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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

MARCH 1960 **VOL. LV. NO. 3**

FIFTY-FIFTH YEAR OF PUBLICATION

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No. 628 ISSUED MONTHLY Registered for Canadian Magazine Post

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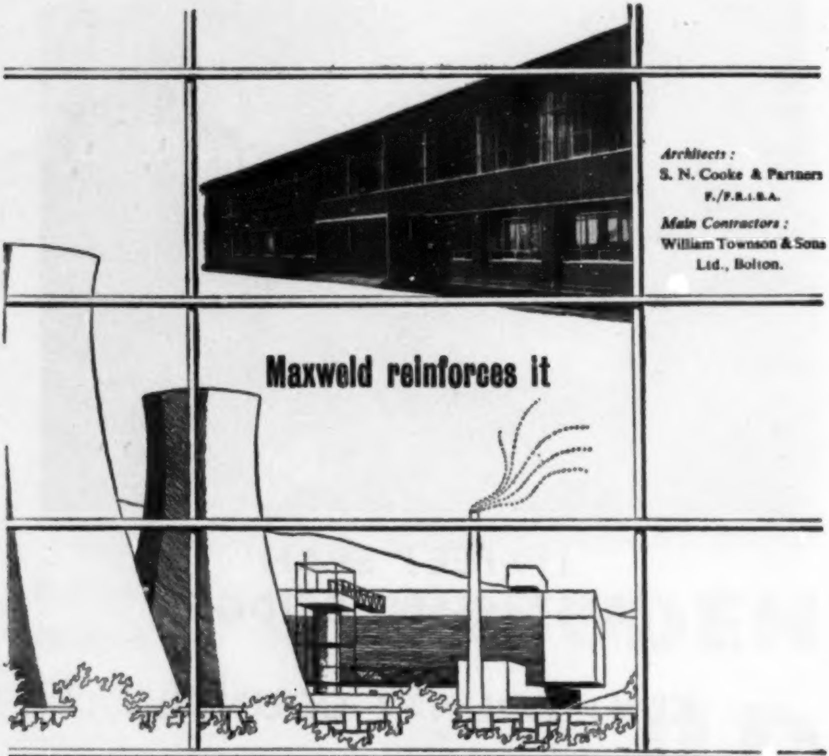
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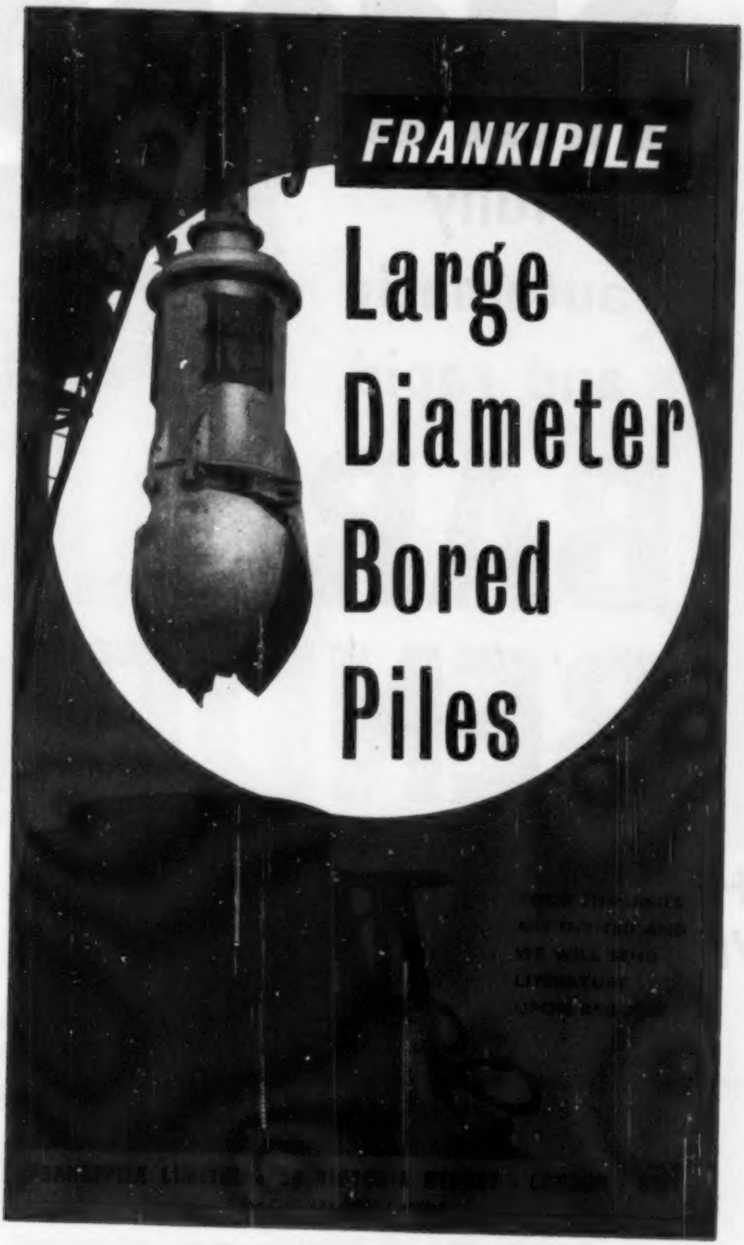
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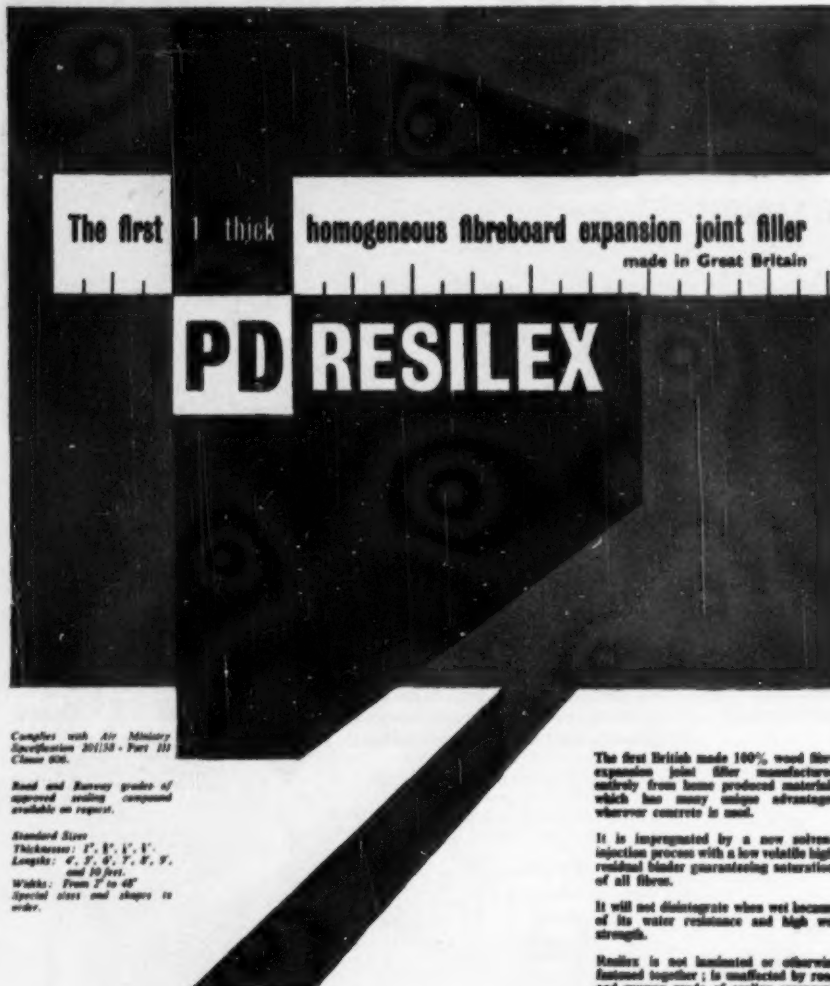
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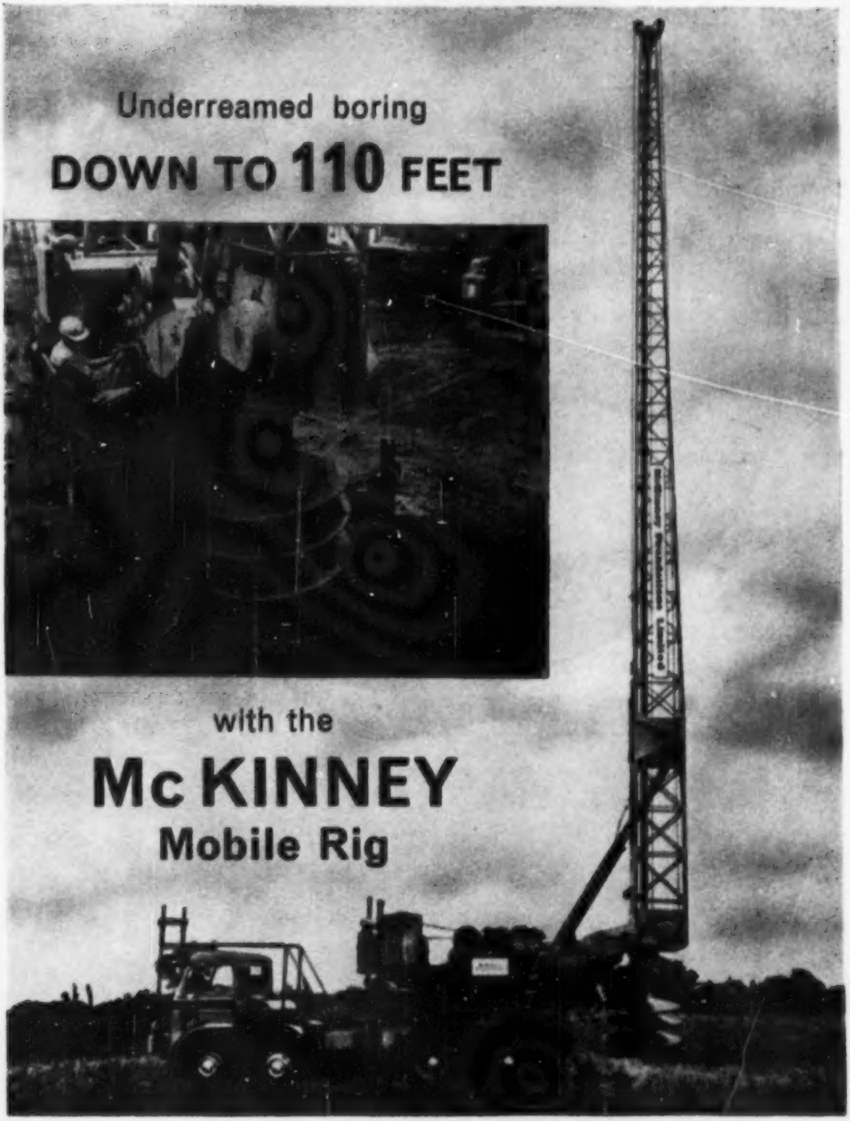
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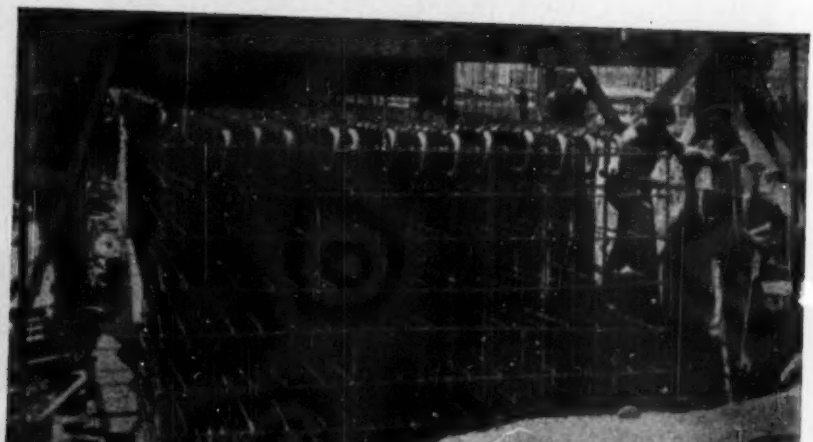

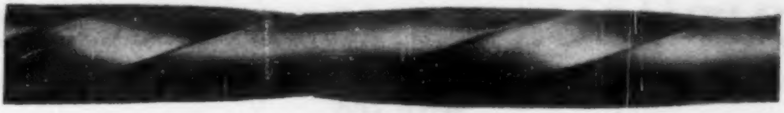

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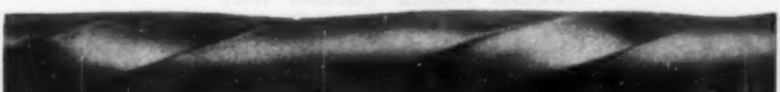

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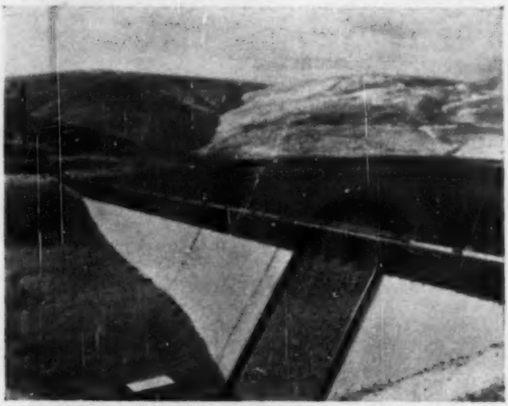

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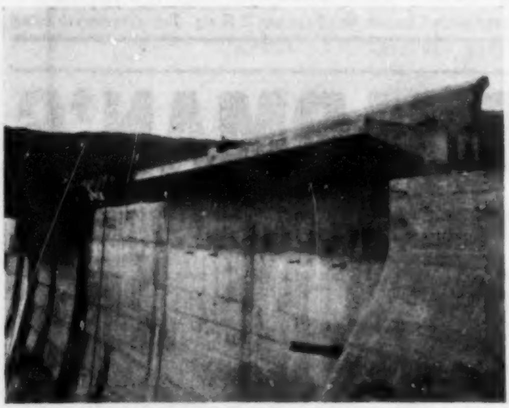

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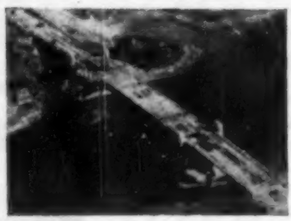

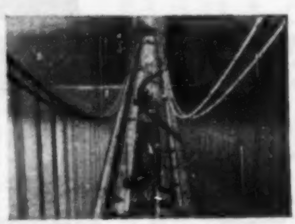

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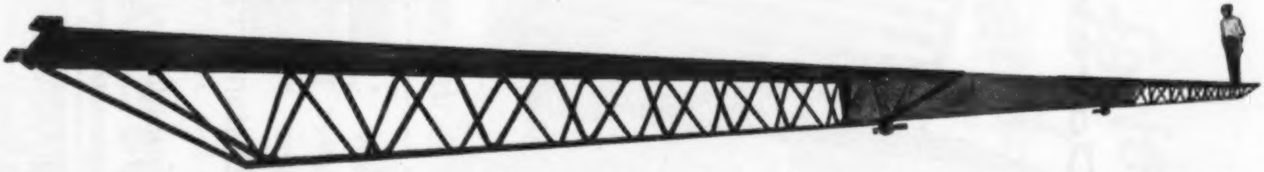

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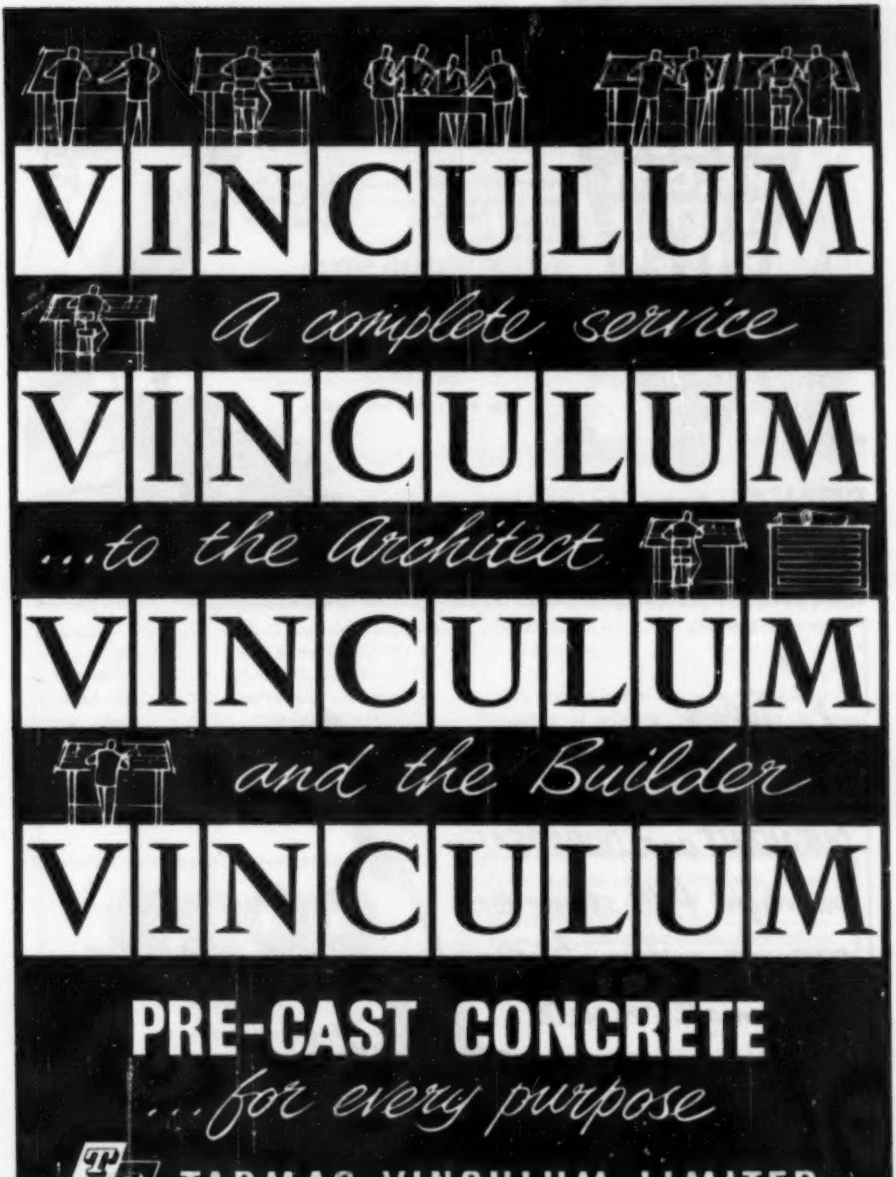

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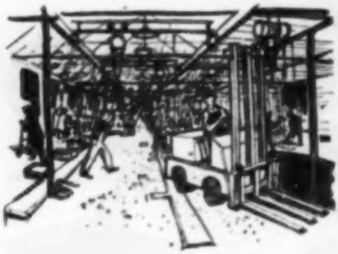

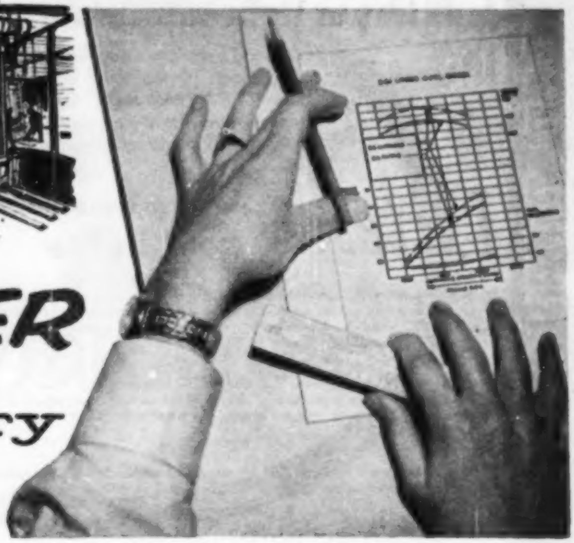

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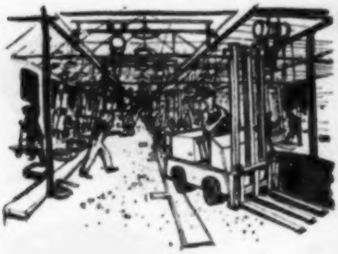

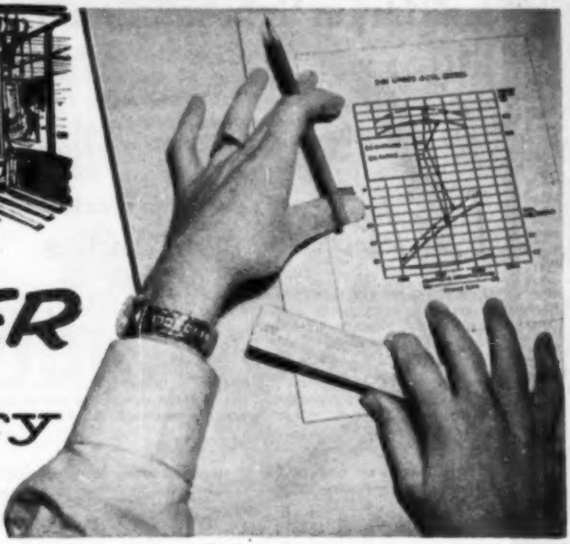

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Bridge Building—the new way

Prestressed concrete bridge on Kumeu-Helensville main highway, North Auckland, New Zealand.

This is one of over fifty prestressed concrete bridges in New Zealand pioneered by B. & B. Concrete Co. Ltd. using Johnsons wire. Varying in span from 30 ft. to 120 ft., they have drastically cut the costs of bridge building in terms of time, money and traffic interference. Speed was essential on the Kumeu Contract. The first of the beams was delivered to site at the end of October 1955, and the bridge had to be open for the Christmas holidays. For further details write for leaflet.

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Pre-stressed Concrete (N.Z.) Ltd.

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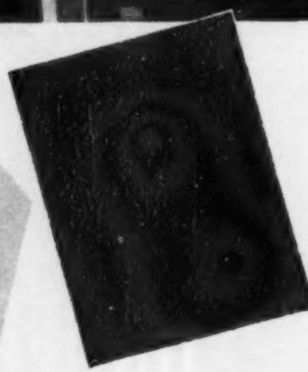

The water tower illustrated has a protective Stic B stone covering stipple finish similar to that shown in the smaller photograph. Treated in this way it is completely protected from the atmosphere and will require no further attention for many years. Stic B can be applied direct to new or old concrete and most other building surfaces. No other material affords such lasting protection, while remaining so easy to apply.

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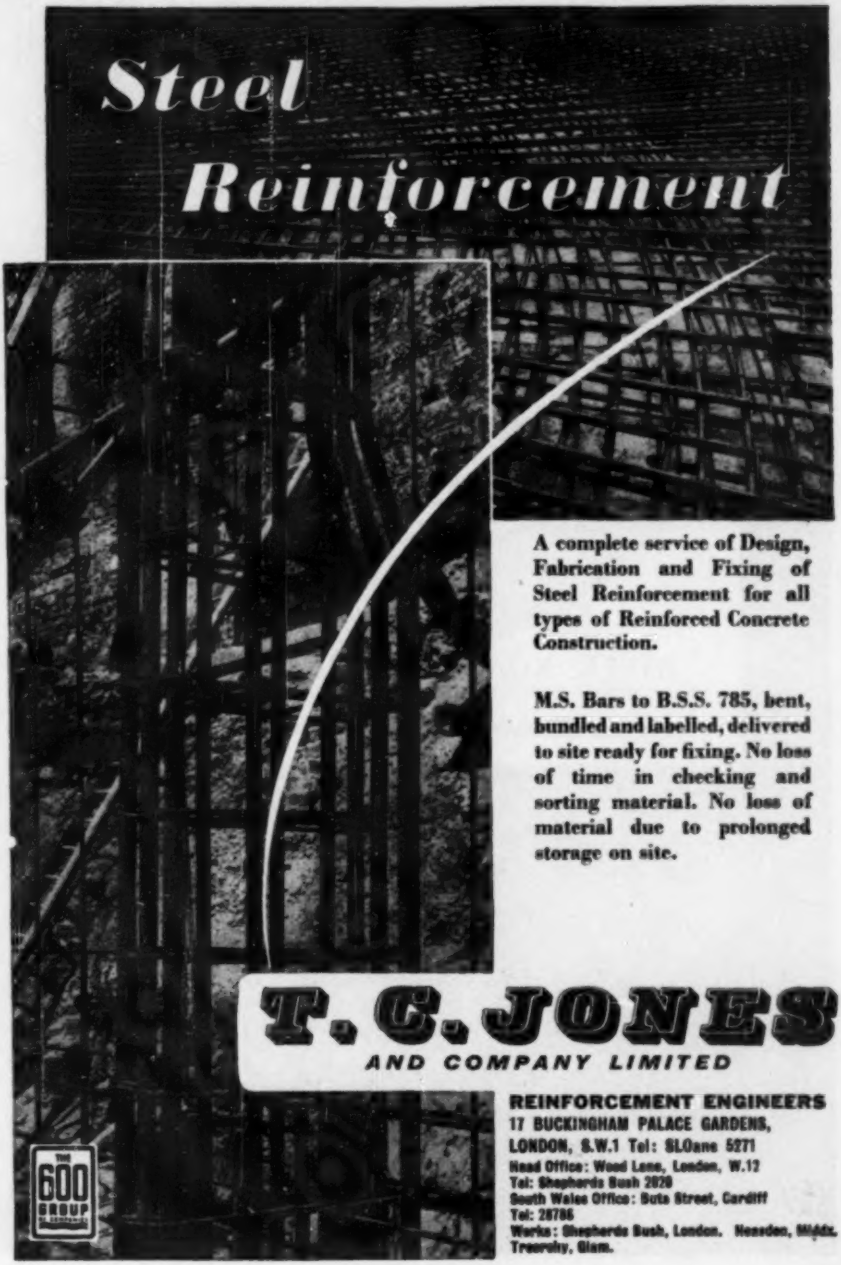
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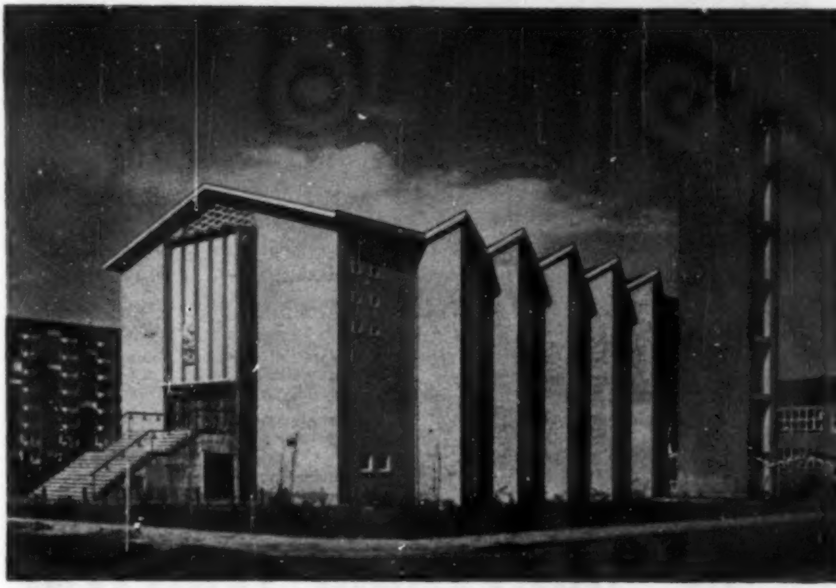

This new church at Vlaardingen near Rotterdam is founded on fifty-nine 17-in. diameter Vibro piles 81 ft. long. The working load per pile is 68 tons, and 19-in. and 24-in. diameter shoes were used to develop the appropriate bearing capacity.

The Vibro process ensures the formation of a pile consisting of dense, compressed concrete which has not been subjected to any driving stresses. The pile is adaptable in length, diameter and size of shoe to suit the ground conditions
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LV, No. 3. London, March, 1960.

EDITORIAL NOTES

"A Word is Wanted": Report on the Competition.

THE misconceptions that can be caused by the use by technologists of wrong and misleading words are well shown in many of the suggestions entered in the competition, announced in this journal in September last, for a word to describe the material (generally steel wire or bars) used to put concrete into a state of compression and thereby produce what is known as prestressed concrete. The term prestressed concrete is itself unsatisfactory because it does not indicate the kind of stress to which the concrete is subjected. Worse was to come when the terms pre-tensioned concrete and post-tensioned concrete were coined to describe concrete which is in fact compressed; in both cases the prefixes indicate the time when the steel—not the concrete—is tensioned. There were more than fifty entrants for the competition and the number of different words submitted exceeded one hundred; some words were submitted by more than one entrant.

The comments in the following are based on the assessor's report of the competition, the purpose of which was to find an acceptable word that would describe the material used to put the concrete into a state of compression and that would indicate this function. Yet nearly a third of the words submitted implied that the prestressing material was stretching rather than compressing the concrete. Among these words are tensor, tenser, tent, tensioning element, tensingforce, tensilers, tensiles, and tensection. One competitor went so far as to suggest the word stretcher to describe the material that compresses the concrete. All these words using or based on the word tension indicate that the competitors who submitted them are the victims of the terms pre-tensioned concrete and post-tensioned concrete, and are in the habit of describing a prestressed member as being tensioned when in fact they well know that it is compressed. Words based on tension would be more aptly used to describe the jack or other means of tensioning the steel rather than the material which applies a compressive force to the concrete. This is an excellent example of the reason for the complaint so often heard that scientists and technologists have invented a language of their own which makes them unintelligible to the rest of the community. It also indicates that men with a knowledge of philology and semantics should be consulted before new technological terms are adopted, particularly when British Standards and codes of practice are being prepared. The unthinking generally

accept without question the terminology of a document having an official appearance, however wrong or foolish it may be.

Much nearer the mark were the competitors who submitted entries based on the word compression. Among such words were compressment, precompressment, compressus, compressing agent, compressifier, comprecement, pressinforcement, and presser, none of which, however, found favour with the assessor. In this group were pressure band (which suggests an element wrapped around the concrete) and pressing member (which indicates an external member applying a pressure to the concrete and which therefore would not be applicable when the force in the steel is transferred to the concrete by bond). Other entries indicating compression were clenchment for the material used to compress the concrete, clenching to describe the application of the prestressing force, and clenched concrete as an alternative to prestressed concrete. The act of clenching is, however, a local operation of closing tightly or grasping firmly, and is in no way similar to the compressing of a concrete member, particularly when the steel is pre-tensioned; there seems to be no analogy between effecting permanent compression and the clenching of teeth or fists—or nails.

Many entries were based on the word stress, including stresser and stressor, stress agent and stressing agent, stresscon, stressment, and—stress conditioner! All these words have the disadvantage that they do not indicate the kind of stress applied to the concrete.

Many competitors attempted to find a word similar to reinforcement, such as preforcement, preinforcement, forcement, and contraforcement, and another competitor suggested the use of preforced concrete in place of prestressed concrete. These words suffer from their similarity to reinforcement, with a consequent risk of confusion.

The assessor makes the following comments on some of the other entries. Bracement and embracement, and fortified (concrete) have well-known meanings, and it does not seem worth while to give them another meaning, and the same applies to lacing, latticing and webbing. Active reinforcement seems to have no meaning. Comprimer is a French verb meaning to compress, and its adoption in other countries as a noun describing a variety of materials in a state of tension does not seem advisable. Forceion carries its own condemnation. Strire is no doubt intended to describe a stretched wire; if this were adopted we should also need strod to describe a stretched rod and strope for a stretched wire rope. Sindon, suggested by the Greek verb meaning to bind together, is at first sight attractive, but in Latin the word sindon means fine linen or lawn. Sennon, sennen, sinnen, and sinnon are old spellings of the modern word sinew, and the use of one of these would seem to be a little far-fetched; besides, a sinew is a tissue uniting a muscle to a bone, and is in tension only when a force is applied to it by a muscle.

As stated in our February number the assessor awarded the prize of £25 to Mr. Paul L. Sowerby, of West Didsbury, Manchester, for the word strictor. This is based on the supine *strictum* of the Latin transitive verb *stringere*, meaning to draw or tie tightly, to press together, to tighten. The word strictor consists of the stem *strict-*, with the suffix *-or* converting it into an agent noun indicating a thing that acts. It seems that of all the suggestions received the word strictor best indicates imparting to a concrete member, by compression, a degree of rigidity which it would not otherwise possess.

Slabs Spanning in Two Directions Designed by the Yield-line Method.

UNIFORMLY-DISTRIBUTED LOADS.

By G. M. MILLS, B.Sc.Tech., A.M.I.C.E., A.M.I.Struct.E.

RECTANGULAR panels of slabs spanning in two directions may, according to British Standard Code of Practice No. 114, be designed by three methods, one of which requires the moment of resistance to be computed by the load-factor method and the determination of the strength by Johansen's yield-line theory.[1] This method may result in considerable saving of material and gives greater flexibility in design than alternative empirical methods. For example, the

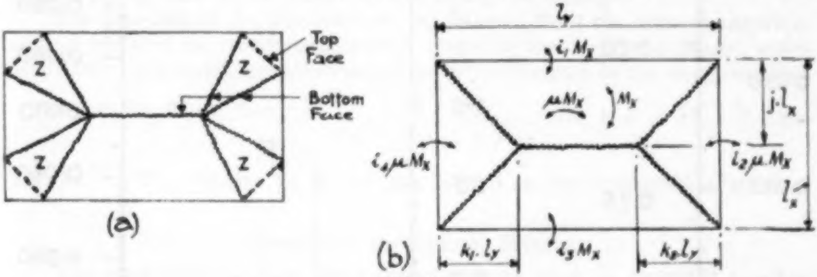


Fig. 1.

moment of resistance over the supports may be arbitrarily equal to the moment of resistance at mid-span, thereby enabling freer use to be made of bent-up bars.

Pattern of Fractures.

For uniformly-distributed loads the pattern of the fractures which develop at failure are normally as in *Fig.* 1a. The provision of additional reinforcement in the top at the corners restricts the formation of the triangular area Z. Sufficient additional reinforcement eliminates this area, thus giving the pattern in *Fig.* 1b. Where the slab is continuous over an edge an additional fracture develops along the edge before complete failure occurs.

The principal notation as given in *Fig.* 1b is self-explanatory.

Formulæ for Design.

There is an infinite number of combinations of reinforcement in the two orthogonal directions and at the supports of a slab which give the same strength. An economical arrangement may be obtained more readily by the use of various devices. Consider the slab in *Fig.* 1b. Assume that there are no shearing forces at the fractures and that the slab is homogeneously reinforced. If $\mu = 1$, Ingerslev showed that, for a uniformly-distributed load w per unit area,

$$M_x = \frac{w l_{xr}^2}{24}\left[\sqrt{3 + \left(\frac{l_{xr}}{l_{yr}}\right)^2} - \frac{l_{xr}}{l_{yr}}\right]^2 \quad . \quad . \quad . \quad (1)$$

in which the reduced lengths of the sides l_{yr} and l_{xr} are

$$\frac{2l_y}{\sqrt{1 + i_2} + \sqrt{1 + i_4}} \quad \text{and} \quad \frac{2l_x}{\sqrt{1 + i_1} + \sqrt{1 + i_3}} \quad \text{respectively.}$$

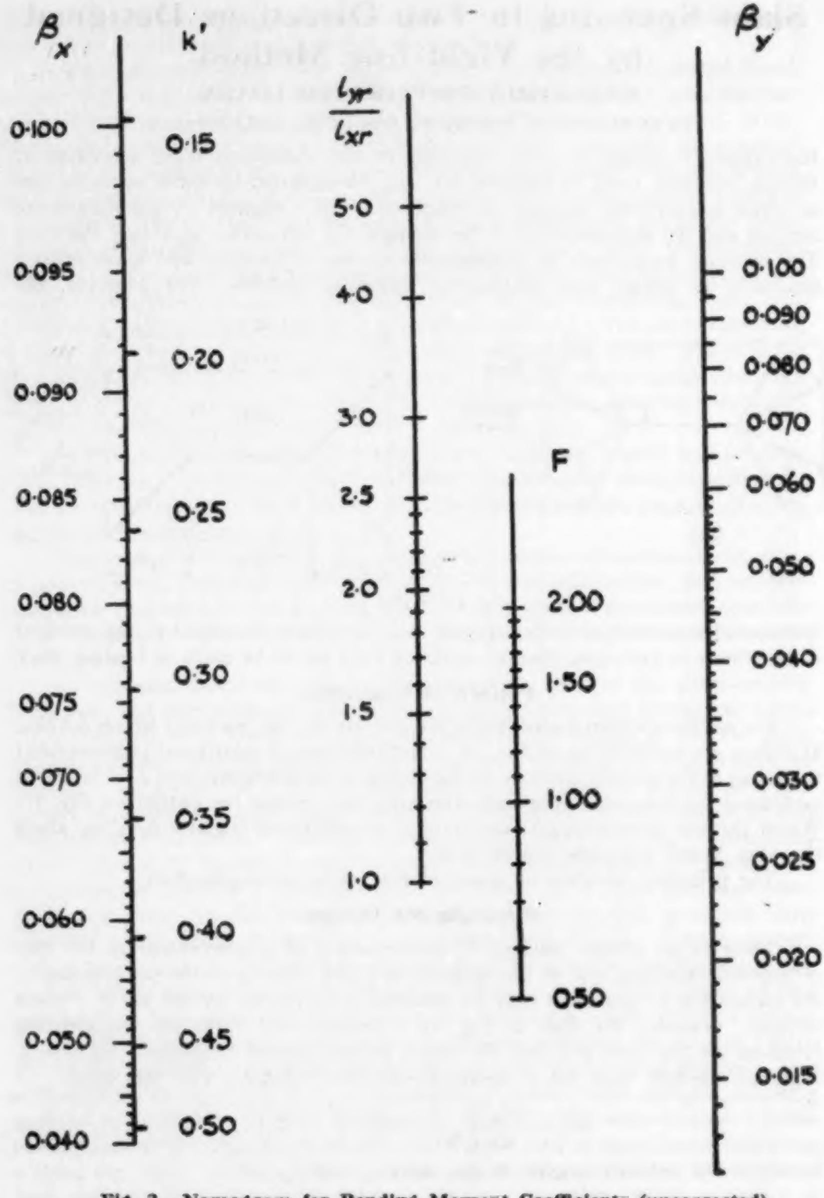


Fig. 2.—Nomogram for Bending-Moment Coefficients (uncorrected).

If μ is not equal to unity, Hognestad[1] shows that the slab may be analysed as an isotropically reinforced slab provided that the linear dimension parallel to the reinforcement giving the yield moment μM_x is divided by $\sqrt{\mu}$ while the load w is unaltered. Substituting in equation (1),

$$M_x = \frac{wl_{xr}^2}{24}\left[\sqrt{3 + \mu\left(\frac{l_{xr}}{l_{yr}}\right)^2} - \frac{l_{xr}}{l_{yr}}\sqrt{\mu}\right]^2 = \beta_x w l_{xr}^2.$$

The tabulation of values of β_x for different values of μ and $\frac{l_{yr}}{l_{xr}}$ are obtained from the nomogram in *Fig.* 2. Values for β_x and β_y are plotted against the ratio of the reduced spans l_{yr}/l_{xr} such that the yield-moment at mid-span of the shorter span is $\beta_x w l_{xr}^2$ and of the larger span is $\beta_y w l_{xr}^2 = \mu\beta_x w l_{xr}^2$.

The positions of the fractures can be obtained from the same assumptions and are required for calculating the load carried by the supporting beams, which may differ considerably from the load allowed if the design is by the elastic method.

It can be shown that j is $\dfrac{\sqrt{1 + i_1}}{\sqrt{1 + i_1} + \sqrt{1 + i_3}}$, and, if $k' = \dfrac{k_1 + k_2}{2}$, then $\mu M_x = \dfrac{w(k' l_{yr})^2}{6}$. Values for k' are also plotted on the nomogram (*Fig.* 2).

Correction for Corner Effects.

The coefficients obtained from the nomogram must be corrected to allow for the formation of additional fractures at the corners of the slab as shown in *Fig.* 1a. This effect reduces the strength of the slab, and the bending moments used in the design must be increased accordingly. Also k' may increase by an amount not exceeding 4 per cent. Let $i_c M_x$ be the yield moment over the support at the corner of the transformed isotropically-reinforced slab. If the values of i for the two adjacent sides are equal and $R_c = \dfrac{i_c + 1}{i + 1} = 1$, Johansen shows that the correction for a square slab is 9·1 per cent. and that this is a maximum. Other percentages for a square slab are 6·8 for 1·1, 4·4 for 1¼, 1·8 for 1½, and 0 for 2. If the transformed slab is not square, the correction for corner effects at a corner where the values of i for two adjacent sides are equal can be obtained from *Fig.* 3 in which the correcting factor is plotted against $\dfrac{l_{yr}}{l_{xr}\sqrt{\mu}}$ for various values of R_c.

It is therefore usually economical to provide some additional reinforcement in the top of the slab at the corners where the corner yield-moment can be obtained without increasing the thickness of the slab, for example in the case of a freely-supported slab. When additional corner reinforcement is provided in one direction only, as at a corner with one adjacent side freely supported and the other continuous, the reduction in the corner effect depends on the angle between the line of the fracture and such additional reinforcement, but cannot be reduced to simple terms. The following empirical rule is suggested for the design of such corners.

Let iM_r be the support yield-moment at the continuous edge of a slab with a yield-moment of M_r at mid-span in a parallel direction and μM_r at mid-span

in a direction at right-angles. If reinforcement is provided in the top at the corners and parallel to the continuous edge to produce a yield-moment of $i\mu M_r$, then $R_e = \frac{1 + i}{1 + vi}$ approximately, in which $v = \frac{l_1}{l_1 + l_2\sqrt{\mu}}$ and l_1 and l_2 are the lengths of the continuous and the freely-supported edges respectively. One-quarter of the correction factor for each such corner should then be obtained from *Fig.* 3.

Economical Amount of Reinforcement.

The condition that $i_1 = i_2 = i_3 = i_4$ is generally convenient for a fully-continuous slab. If, also, the area of reinforcement in each orthogonal direction is proportional to the corresponding design moment at mid-span, the total weight of reinforcement is proportional to $M_x + \mu M_x = S$ such that

$$S = (1 + \mu)\left[\sqrt{3 + \left(\frac{l_{xr}}{l_{yr}}\right)^2 \mu} - \frac{l_{xr}}{l_{yr}}\sqrt{\mu}\right]^2 C'\frac{wl_{xr}^2}{24},$$

in which $C' = 1 +$ (correction factor for corner effects). The correction factor is a function of μ, but it is reasonable to consider C' as relatively constant. To find the minimum weight of reinforcement, from $\frac{dS}{d\mu} = 0$, $\mu = \frac{\left(\frac{l_{xr}}{l_{yr}}\right)^2}{3 - 2\left(\frac{l_{xr}}{l_{yr}}\right)^2}$. It will be found that the lines on the nomogram giving the values in *Table* I all pass through unity on line F. Similarly it can be shown that if $\mu \times$ (weight of reinforcement in the M_x direction) $\div$ (weight of reinforcement in the μM_x direction) $= F$, then

TABLE I.

$\frac{l_{yr}}{l_{xr}}$	1	1·1	1·2	1·3	1·4	1·5	1·75	2
μ	1·000	0·610	0·430	0·325	0·260	0·210	0·140	0·100
β_x	0·042	0·056	0·067	0·076	0·083	0·088	0·098	0·103
β_y	0·042	0·035	0·029	0·025	0·022	0·019	0·014	0·011

the most economical distribution of reinforcement occurs when the lines on the nomogram cut line F at the corresponding ratio. It is assumed in the foregoing that the area of reinforcement is proportional to the yield-moment. This is not correct, but the error is negligible.

Position of Reinforcement.

The reinforcement should be designed so that the elastic moments developed are approximately proportional to the design yield-moments. This method not only prevents yielding of reinforcement at working loads but also reduces the widths of the cracks on the face of the slab in tension. For the same reason it may be advisable to adopt a minimum value for the coefficient for the bending moment on the longer span.

Reinforcement to give the design moment of resistance must be provided across all normal lines of fracture. Reference to *Figs.* 1a and 1b indicates the critical sections which must be reinforced.

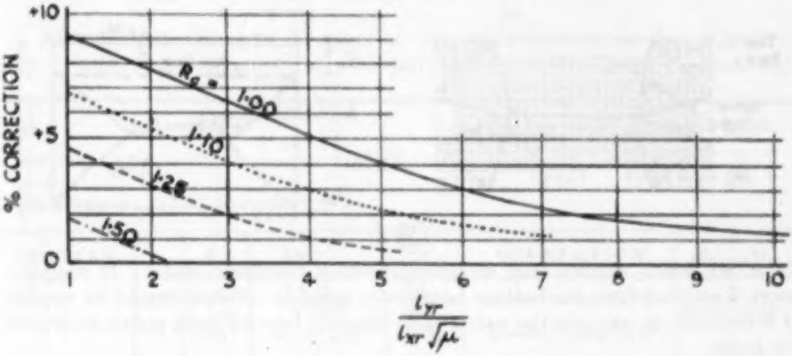


Fig. 3.—Correction for Corner Effects.

Sufficient reinforcement must be provided at other positions to prevent fractures developing across what would otherwise be weakly-reinforced sections and so lead to the collapse of the slab at a load less than the ultimate design load. For example, if reinforcement is omitted from the bottom of the edge strips parallel to the sides in the manner recommended in B.S. Code No. 114, Clause 314b (ii), a possible pattern of fractures is shown in *Fig.* 4a and may result in a lower strength than a homogeneously reinforced slab.

Table II gives typical results obtained by analysis of slabs continuous on four sides in which the moment at the support is equal to the parallel mid-span moment of resistance. The distribution of the reinforcement is "economical".

TABLE II.—VALUES OF $\frac{wl_x^2}{M_x}$.

$\frac{l_y}{l_x}$	Homogeneous	Edge strip $\frac{l_x}{12}$ wide	Edge strip $\frac{l_x}{8}$ wide	
		Corner panel $\frac{l_x}{4} \times \frac{l_x}{4}$	Corner panel $\frac{l_x}{5} \times \frac{l_x}{5}$	Corner panel $\frac{l_x}{3} \times \frac{l_x}{3}$
1·0	44·0	43·8	39·9	46·3 } Fracture pattern as in *Fig.* 4a.
1·2	27·7	27·3		
1·4	22·6	22·3	20·8	23·5 } Fracture pattern as in *Fig.* 4a.
1·6	20·3	20·0		

The determination of the positions at which bars comprising the reinforcement over the supports may end also depends on the need to avoid the development of a pattern of fractures resulting in reduced strength. Consider the slab in *Fig.* 4b which is continuous along one side only. Assume that a fracture develops in the top face a distance nl_x from the continuous edge where there is no reinforcement in the top.

If the strength in this condition is the same as when the fracture occurs along the support, then $l_{xr} = l_x - nl_x$. The length of the reinforcement over the support is therefore dependent on the ratio i. With continuity over two opposite edges, a similar equation determines the minimum value for n. Values of n are given in

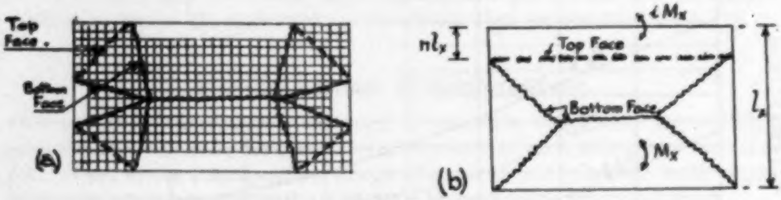


Fig. 4.

Table III which applies only to homogeneously reinforced slabs. If reinforcement is omitted from the bottom in the edge strips, a correction must be applied. It is essential to continue the bars a short distance beyond these points to provide for bond.

TABLE III.—MINIMUM VALUES OF n.

Value of i	1	$1\frac{1}{4}$	$1\frac{1}{2}$	2
Continuous over one side . . .	0·18	0·20	0·23	0·27
Continuous over two opposite sides .	0·15	0·17	0·19	0·22

Similarly, the positions at which a proportion of the reinforcement in the top may be bent down into the bottom of the slab can be determined. For example, reinforcement corresponding to $0{\cdot}5\ M_x$ may be bent down at $n = 0{\cdot}1$ for any value of i if M_x is the corresponding moment of resistance at mid-span.

Summarising the foregoing, reinforcement may be omitted from the bottom of the edge strips of width $l_x/12$ and parallel to the adjacent side subject to the provision of distribution bars and reinforcement in corner panels on sides equal to $l_x/4$. If such a system is adopted the design moments must be increased by 2 per cent. (see *Table* II) and the length of reinforcement over the support by ($0{\cdot}02 \times$ span).

As fractures along continuous supports will occur at the face of the supporting beams, the effective span may be assumed to be the distance between the faces of the beams, and by so doing the design moments may be reduced by as much as 10 per cent. If a slab is not continuous over a support but is constructed monolithically with the support, the ultimate restraining moment afforded by such a connection can be calculated and this may be taken into account in determining the greatest load that can be carried by the slab.

Example.

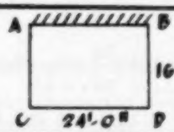
A panel of slab 16 ft. by 24 ft. which is continuous over one longer side and freely-supported along the other three sides is to carry 120 lb. per square foot in addition to its weight. In this case the ratio of the elastic moment at the support

to that at mid-span (using Dr. Marcus's method as modified by Professor B. Loser) depends on whether the panel is one of a row or one of two panels and would be either 1·6 or 1·9 respectively. Therefore i is preferably between 1·5 and 2·0.

If $i = 1{\cdot}5$, $l_{xr} = \dfrac{2 \times 16}{1 + \sqrt{2{\cdot}5}} = 12{\cdot}44$ ft., and $\dfrac{l_{yr}}{l_{xr}} = 1{\cdot}93$.

Assume that the area in plan of the reinforcement at the support is $0{\cdot}25 \times$ (area of reinforcement at mid-span), then $F = 1 + (0{\cdot}25 \times 1{\cdot}5) = 1{\cdot}37$.

CONCRETE
1:2:4
(1000 lb. per sq. in.)



Loading. Imposed 120 lb. per sq. ft.
5" Slab 63 " " " "
Total 183 " " " "

SHORTER SPAN

i	F	l_{xr}	l_{yr}/l_{xr}	CORRECTION	β_x	μ
1·5	$1+\frac{1\cdot5}{4}$ $=1{\cdot}37$	$\frac{2\times16}{1+\sqrt{2\cdot5}}$ $=12{\cdot}4$ ft.	$\frac{24}{12\cdot4}=$ 1·93	Edge Strips: 2% Corners: (say) 2%	0·095	0·23

$M_x = 0{\cdot}095 \times 1{\cdot}04 \times 183 \times 12{\cdot}4^2 = 2780$ ft-lb.

Bar dia. $=\frac{5}{8}$", cover $=\frac{5}{8}$", $d_1 = 4{\cdot}06$. $M_r = \dfrac{250 \times 4{\cdot}06^2 \times 12}{12} = 4120$ ft-lb.

$\dfrac{M}{bd_1^2} = \dfrac{2780}{4{\cdot}06^2} = 168$; Reinf^t. $= 0{\cdot}97\%$; $A_{st} = \dfrac{0{\cdot}97}{100} \times 12 \times 4{\cdot}06 = 0{\cdot}47$ sq. in.

REINFORCEMENT	% (ACTUAL)	$\frac{M}{bd_1^2}$	
Bottom: $\frac{5}{8}$" at 8" c/c.	0·95	165	M_r (actual) $= 16 \times 4{\cdot}06^2$ $= 2680$ ft-lb.
Top A-B { $\frac{5}{8}$" at 8" c/c. / $\frac{1}{2}$" at 8" c/c. }	1·57	240	
Corner Panels Top C & D } $\frac{1}{2}$" at 12" c/c.	0·40	76	i actual $= \frac{240}{163} = 1{\cdot}47$

LONGER SPAN

$V = \dfrac{24}{24 + 16\sqrt{0{\cdot}23}} = 0{\cdot}76$

l_{xr}	l_{yr}/l_{xr}	CORRECTION	β_x ACTUAL	β_y
$\frac{2\times16}{1+\sqrt{2\cdot47}}$ $=12{\cdot}45$ ft.	$\frac{24}{12\cdot45}$ $=1{\cdot}93$	C-D $\frac{l_{yr}}{l_{xr}\sqrt{\mu}} = \frac{1\cdot93}{\sqrt{0\cdot23}} = 4$ If 50% M_r, correction = 0. A-B Provide 1·47 M_y $R_c = \frac{1+1\cdot47}{1+0\cdot76\times1\cdot47} = 1{\cdot}16$ Correction $= 2 \times \frac{1}{4} \times 2\% = 1\%$	$\frac{2680}{183\times1\cdot03\times12\cdot45^2}$ $= 0{\cdot}092$	·024

$M_y = 0{\cdot}024 \times 1{\cdot}03 \times 183 \times 12{\cdot}45^2 = 700$ ft-lb.

Bar dia. $= \frac{3}{8}$". Cover $= 1\frac{1}{4}$". $d_1 = 3{\cdot}56$.

$\dfrac{M}{bd_1^2} = \dfrac{700}{3{\cdot}56^2} = 55$; Reinf^t $= 0{\cdot}29\%$; $A_{st} = \dfrac{0{\cdot}29}{100} \times 12 \times 3{\cdot}56 = 0{\cdot}124$ sq. in.

Bottom: $\frac{3}{8}$" at $10\frac{1}{2}$" c/c.
Corner Panels: Top C & D: $\frac{3}{8}$" at 12" c/c. Top A & B: $\frac{3}{8}$" at 6" c/c.
Distrib^{tion}. bars in top A-B: $\frac{3}{8}$" at 12" c/c.

Fig. 5.

From the nomogram in *Fig.* 2, $\beta_x = 0{\cdot}095$ and $\beta_y = 0{\cdot}022$ before correction, giving $\mu = 0{\cdot}23$ and $\dfrac{l_{yr}}{l_{xr}\sqrt{\mu}} = 4$. From *Fig.* 3, for the correction to be zero, R_c must be about 1·4. This value can be obtained at the two corners with both edges freely supported if reinforcement is provided in the top to provide, say, in each direction, half the corresponding moment at mid-span. If the empirical method is adopted for the other two corners and 1·5 × (area of the parallel mid-span reinforcement) is provided at right-angles to the reinforcement in the top over the support,

$$R_c = \frac{1 + 1{\cdot}5}{1 + 1{\cdot}5v} \text{ in which } v = \frac{24}{24 + 16\sqrt{0{\cdot}23}} = 0{\cdot}76;$$

Therefore $R_c = 1{\cdot}17$ and the correction is $2 \times \frac{1}{4} \times 2$ per cent. = 1 per cent. Therefore $M_x = 0{\cdot}096 \times 12{\cdot}44^2 \times w$ ft.-lb. With a 5-in. slab the total load is 120 + 63 = 183 lb. per square foot, and M_x is about 2700 ft.-lb.

If $i = 2{\cdot}0$, $l_{xr} = \dfrac{2 \times 16}{1 + \sqrt{3}} = 12{\cdot}1$ ft., and $\dfrac{l_{yr}}{l_{xr}} = 1{\cdot}98$. The area over which the reinforcement at the support must be provided is increased to, say, 0·3 × (area of mid-span reinforcement) as shown in *Table* III. Therefore

$$F = 1 + (0{\cdot}30 \times 2) = 1{\cdot}6.$$

Proceeding as before, $\beta_x = 0{\cdot}092$ and $\beta_y = 0{\cdot}027$ before correction, and the correction is again found to be 1 per cent. Therefore

$$M_x = 0{\cdot}093 \times 12{\cdot}1^2 \times 183 = 2500 \text{ ft.-lb.},$$

and the moment at the support is 5000 ft.-lb. If the permissible compressive stress in the concrete is 1000 lb. per square inch, the moment of resistance, calculated by the load-factor method, of a 5-in. slab with $\frac{3}{8}$-in. bars and $\frac{5}{8}$-in. cover of concrete is only 4120 ft.-lb. By reducing β_x to 0·078 and increasing β_y to 0·053, $M_x = 2060$ ft.-lb. and the moment at the support is 4120 ft.-lb. A comparison of the foregoing results with the bending moments (in ft.-lb. per foot width) calculated by use of the coefficients in *Table* 17 of B.S. Code No. 114 is as follows.

Bending moment		Load-factor Method $i = 1{\cdot}5$	Load-factor Method $i = 2{\cdot}0$	Elastic Theory. Coefficients from Code No. 114 $l_x = 16{\cdot}5$ ft.
Support:	iM_x	4000	4120	4150
Midspan of l_x:	M_x	2670	2060	3110
Midspan of l_y:	$M_y = \mu M_x$	620	1350	2150

The first trial, that is with $i = 1{\cdot}5$, gives the most economical design and the saving in reinforcement in a 5-in. slab compared with the elastic method is about 20 per cent.

The foregoing calculation shows the use of the nomogram and the graph for finding the correction factor; in practice, it may be more convenient to base the calculations on a selected arrangement of the reinforcement across the shorter span. A typical calculation sheet is shown in *Fig.* 5 in which moments of resistance are obtained from load-factor tables.[2] For the short span a preliminary calculation

is carried out to determine the approximate "economical" moment of resistance as a guide to selecting suitable reinforcement. Using the value of i obtained thus, the reinforcement across the longer span is obtained by means of a more accurate calculation.

(1).—E. HOGNESTAD, "Yield-line Theory for the Ultimate Flexural Strength of Reinforced Concrete Slabs". Journal of Am. Concrete Inst., March 1953.

(2).—J. S. COHEN, "Tables for the Design of Beams and Slabs". Concrete Publications Ltd.

Residential Flats 97 ft. and 106 ft. High.

THE residential flats illustrated below, which have been erected at Shoot-up Hill for the Willesden Borough Council, comprise two structures 97 ft. high each of eleven stories and one structure 106 ft. high of twelve stories. The plan of each block is in the form of two rectangles one of which is about 63 ft. by 28 ft. and the other about 45 ft. by 35 ft. The two parts are connected by a structure, containing the lift, about 22 ft. by 8 ft. The main roof is surmounted by tank-rooms, the lift-motor room, and a penthouse.

The structures have strip footings, and the floors, roofs, stairs, walls enclosing the stairs, and the lift shafts, some external walls, and the internal cross-walls are of reinforced concrete. The external concrete walls are faced with brick or faience. The remaining external walls are of brickwork with an inner lining of clinker-concrete slabs. The internal partitions are also of clinker-concrete slabs. The beams, staircases, and balconies were precast.

The internal and external reinforced concrete walls, which were cast in place, carry the weight of, and loads on, the building to the foundation. The outer faces of the external walls were coated with a waterproofing bituminous compound before building the brick facing. There are also some single-story buildings.

The cost of the work is about £400,000.

The architects are Messrs. Emberton, Franck & Tardrew. The designers of the reinforced concrete work and the contractors were Messrs. Wates, Ltd.

Book Reviews.

"Proceedings of a Symposium on the Strength of Concrete Structures." Obtainable from Concrete Publications, Ltd. Price by post £5; in U.S.A. and Canada, 20 dollars.

THE proceedings of the Symposium on the Strength of Concrete Structures, organised by the Cement and Concrete Association and held in London in May, 1956, have now been published in a volume containing the seventeen papers, a general report and a record of the discussions. The subjects and authors of the papers are:

"Some Results of the Theory of Probability in the Estimation of Design Loads", by M. R. Horne. "Determination of the Design Factor for Reinforced Concrete Structures", by A. I. Johnson. "Current Trends in the Specification of Structural Safety", by Professor Sir Alfred G. Pugsley, O.B.E.

"Strength of Singly-reinforced Beams in Bending", by A. H. Mattock. "Strength of Concrete Members in Combined Bending and Torsion", by S. Armstrong. "Strength of Prestressed Concrete Members", by Professor C. P. Siess. "Moment Redistribution in Continuous Beams Reinforced with Plain and Deformed Bars", by K. Hajnal-Kónyi and H. E. Lewis. "Failure of Concrete under Compound Stress", by A. J. Harris.

"Ultimate-load Design of Reinforced and Prestressed Concrete Frames", by Professor A. L. L. Baker. "Strength of Statically-Indeterminate Prestressed Concrete Structures", by Y. Guyon. "Strength of Prestressed Concrete Continuous Beams and Simple Plain Frames", by P. B. Morice and H. E. Lewis.

"Strength of Concrete Walls under Axial and Eccentric Loads", by A. E. Seddon. "Strength of Concrete Members under Dynamic Loading", by S. C. C. Bate. "Strength of Simply Supported Slab Bridges subjected to Concentrated Loads", by P. B. Morice and G. C. Reynolds.

"Ultimate Strength of Reinforced Concrete in American Design Practice", by E. Hognestad. "Design of Reinforced Concrete Members", by A. Aas-Jakobsen. "Load-factor Design in Building Regulations: Future British Practice", by D. D. Matthews.

"Examples of the Design of Reinforced Concrete Buildings in accordance with the British Standard Codes." By Charles E. Reynolds. (London: Concrete Publications, Ltd. 2nd edition. 1959. Price 12*s*. 6*d*.)

NEW features in the revised British Standard Code No. 114 (1957), "The Structural Use of Normal Reinforced Concrete in Buildings", include the load-factor method of design of slabs, beams, and columns subjected to bending, and the analysis of flat slabs as parts of a frame. These matters and other unaltered or revised recommendations of the B.S. codes relating to the design of reinforced concrete buildings are dealt with in the second edition of this book, which has been almost completely rewritten and includes the revised notation. As in the previous edition the book is in two parts.

In the first part design data are considered in the same sequence as in design calculations, namely, loads, bending moments and shearing forces, stresses, resistance to bending and shearing, and bond. The structural parts of a building are then dealt with in the order in which design proceeds, namely, slabs, beams, columns and load-bearing walls, stairs, basements, and foundations. The method of moment-distribution applied to continuous beams and the design of hollow-block slabs and precast floors are included as in the previous edition, and among the new matter there is a consideration of concentrated loads on slabs spanning in one or two directions. More than sixty tables and numerous diagrams are provided to aid quick design.

In the second part the complete design of a typical multiple-story building is given in the form of calculations and drawings showing the arrangement of the members and the reinforcement. Alternative designs of beam-and-slab and flat-slab floors are given. Slabs spanning in two directions with and without the corners held down, cantilevered slabs, columns (with and without the effects of wind being taken into account), and the effect of bending in two planes are considered. The building is planned to incorporate as many of the recommendations of the British Standard Codes as is practicable.

Prestressed Road Bridge of 110-ft. Span.

THE road bridge (*Fig.* 1) erected recently over the railway between Bickley and St. Mary Cray, Kent, has a span of 110 ft. and is one of the largest freely-supported prestressed concrete bridges of its kind in Britain. The deck (*Figs.* 3 and 4) comprises twelve longitudinal prestressed beams of tee-section on the top flange of which there is a layer of plain concrete and 2 in. of hot-rolled asphalt. Vertical slabs are provided at 12 ft. centres between the flanges. The overall width of the bridge, which is on a slight skew in relation to the railway, is 47 ft. 7 in.; the carriageway is 33 ft. wide and there is a 5-ft. footpath at each side with provision for pipes and services below. The parapets are steel hand-rails. Tubular guard-rails are provided between the footpaths and the carriageway.

Fig. 1.

Each beam is supported on concrete hinges (*Fig.* 5). At one end there is a simple hinge, and at the other end a rocker-type hinge. Each abutment is a continuous pile-cap 10 ft. wide supported on twenty-two cast-in-place piles, 14 in. in diameter and 30 ft. long, bored in the sides of the cutting.

The bridge, which is designed to carry the ordinary load specified by the Ministry of Transport and Civil Aviation, replaces a brick structure of three arches. A part of the bridge comprising five beams was constructed first and road traffic was diverted to this part while the old structure was demolished (*Fig.* 2). The second part of the new bridge, comprising seven beams, was then constructed.

Beams.

The beams were first designed to be precast at the site in one length of about 113 ft., but owing to the restricted space they were each precast in five parts at a factory. The three longer intermediate parts are each 25 ft. long. The vertical slabs between the flanges are 9 in. thick and are monolithic with the beams. The width of the beams (including the vertical slabs) as cast is such that there were gaps of 5 in. between adjacent beams, which were filled with high-alumina cement concrete. Before it was erected each beam was prestressed with eleven cables each containing twelve 0·276-in. wires which were tensioned one at a time. The anchor-plates at the end of a beam are shown in *Fig.* 7. The deck was prestressed transversely, as shown in *Fig.* 4. The concrete in the beams had a water-cement ratio of 0·40 and a crushing strength of not less than 7250 lb. per square inch at 28 days. The strength at seven days or at the time of prestressing was not less than 5000 lb. per square inch.

Fig. 2.—The Old Bridge during Demolition.

Fig. 3.

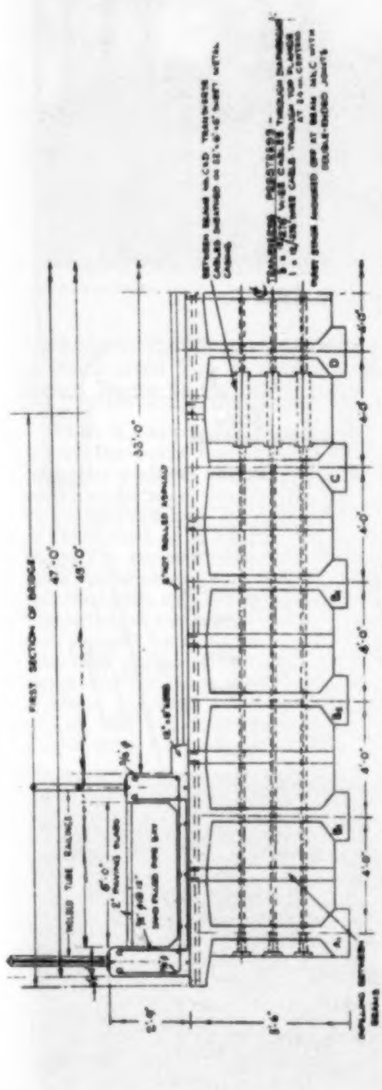


Fig. 4.—Half Cross Section AA (*see Fig.* 3) of Deck.

Hinges.

The hinges (*Fig.* 5) are of the Freyssinet type as described by M. Y. Guyon in a paper entitled "Long-span Prestressed Concrete Bridges Constructed by the Freyssinet System" (Proceedings of the Institution of Civil Engineers, May, 1957). Double throatings are provided in the rocker hinges and a single throating in the fixed hinges. The hinges, which are lightly reinforced, are of high-alumina cement concrete cast in place. The beam bears directly on the top of the hinge from which a stub projects and registers with a recess in an iron ring 1¼ in. diameter and 1 in. deep built into the soffit of the beam.

Erection.

The erection of the beams was hampered by the restricted working space on a busy road which had to be kept open for traffic, and the few hours each week during which it was permissible to work over the tracks. Each part of a beam was brought to the site on a lorry which backed under a steel gantry (*Fig.* 8) at the side of the road at one end of the bridge. The beam was lifted by two 7-ton lifting gears suspended from the gantry, and after the lorry had moved away it was lowered on to a bogie on a rail track and transported to the assembly bay (*Fig.* 9). Here the parts were arranged to form complete beams and supported on stools. Each stool comprised a concrete block surmounted by two steel beams. Steel packings were used to obtain the correct height; this was important, because it was necessary for the 2-in. ducts for the three transverse prestressing cables in each vertical slab to register exactly. It was also necessary to align the parts longitudinally to within ⅛ in. to ensure that the ducts (which are at 2 ft. centres) in the top flange were in alignment with the ducts in adjacent beams.

The four joints between the five parts comprising a beam are about 1 in. wide and were packed with dry mortar. Short lengths of inflatable rubber tube were inserted across the joints in the ducts for the longitudinal cables to prevent mortar gaining access to the ducts. Four days after making the joints, when the strength of the mortar was about 5000 lb. per square inch, the beams were prestressed, an operation occupying about two days. The hogging due to the prestressing raised

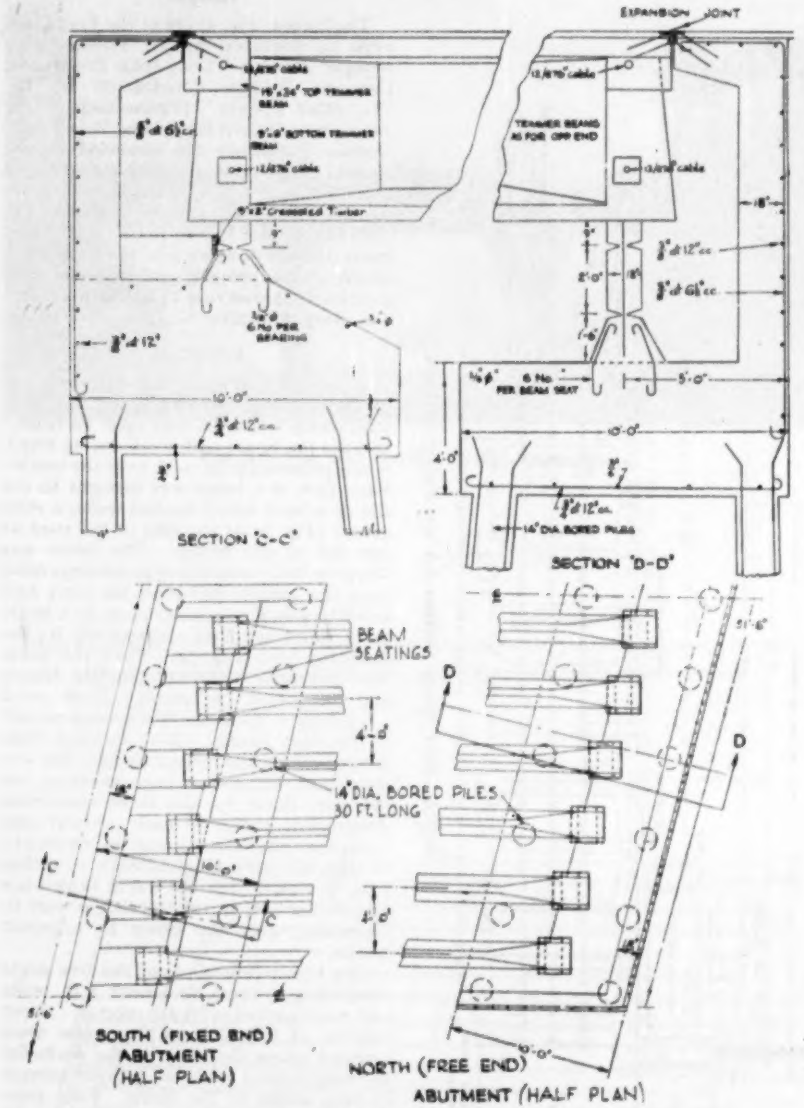


Fig. 5.—Details of Abutments.

Fig. 6.—Placing Beam in Position.

the beam off the stools, the steel bearings of which could then be removed and so permit bogies to be placed under the beam.

When it was possible to work over the tracks the beam was drawn on to the bridge by a winch. A steel gantry of span sufficient to cover the first five beams to be erected had been installed at each end of the bridge, as is seen in *Fig. 6*. Steel slings (*Fig.* 10) suspended from the gantries were placed around each end of the beam, and the beam, which weighs about 50 tons, was lifted off the bogies, moved laterally, and lowered into position. The concrete was then placed in the gap between the beam and the beam placed previously.

When the first five beams had been placed and the intervening gaps filled, cables were inserted in the ducts in the flanges and the vertical slabs and tensioned, thereby prestressing transversely the first part of the bridge.

The road surface on this part was then laid and traffic diverted from the old bridge. The steel gantries were dismantled and, after demolition of the old bridge, were re-erected for erecting the remaining seven beams. When the first two of these beams had been laid, temporary transverse prestressing was carried out to tie these beams to the first part so that they could be used as a platform on to which the remaining beams could be drawn from the assembly bay. This procedure enabled the entire width of the

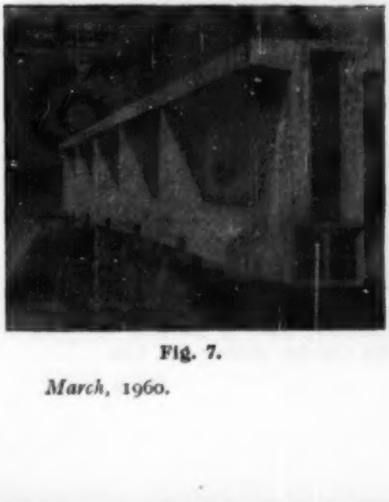


Fig. 7.

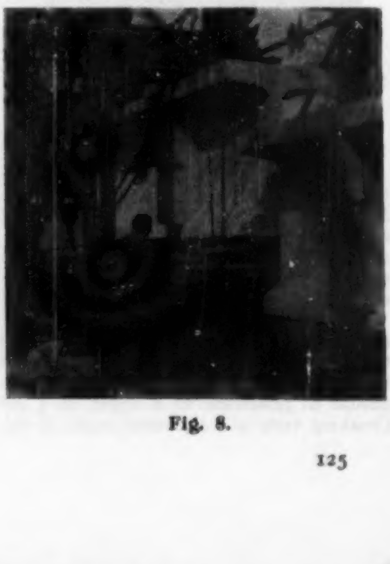


Fig. 8.

first part to be used by traffic. The transverse prestressing was continued by attaching, by means of a double-ended joint, an additional length of cable to that already tensioned.

Because of the position of the inclined struts of the gantries, the outer beam could not be lowered into its final position directly. Therefore after the tenth beam had been placed, the outer (or twelfth) beam was lowered into the position of the eleventh beam and then moved laterally to its correct position. The eleventh beam was then placed in the space between the outer and tenth beams. The gaps were filled and the transverse prestressing continued across the entire width of the bridge.

The bridge was designed by the late Mr. J. A. Riccomini, Bridge Engineer, Roads Department, Kent County Council, and was erected under the supervision of Mr. E. W. H. Vallis, at that time County Surveyor. This article has been prepared with the co-operation of the present County Surveyor, Mr. H. Bowdler, M.I.C.E. The contractors were Messrs. John Cochrane & Sons, Ltd., to whom Messrs. Bernard Clarke & Partners acted as consulting engineers. The precast blocks were made by Concrete, Ltd. The beams were prestressed by the Gifford-Udall system. The piles were supplied by the Cementation Co., Ltd.

Fig. 9.

Fig. 10.

Non-destructive Method of Testing the Strength of Lightweight Concrete.

In the course of a paper presented to the American Society for Testing Materials in June, 1959, Mr. I. A. Benjamin and Mr. G. D. Ratcliff describe the results of an investigation on the use of a Proctor needle to ascertain the strength of concrete with a density of less than 40 lb. per cubic foot.

The procedure is to apply a Proctor needle to the concrete at any time more than six hours after it has been cast and to record the force required to cause the needle to penetrate to a depth of ¼ in. Crushing tests of 2-in. cubes made of the same concrete showed that there was a consistent relation between the force applied to the needle and the compressive strength of the cubes at ages up to more than 21 days. Full details of the investigation are available from the Society (1916 Race Street, Philadelphia, Pa., U.S.A.) in the form of a preprint of a paper to be published later in the Bulletin of the Society.

A Low-cost Warehouse.

Mr. S. Stothert, who was associated with the design of the new warehouse described on page 40 of our number for January, 1960, is the staff architect of the Clayton Aniline Co., Ltd.

Comparison of Analyses of Helical Stairs.

By V. A. MORGAN, M.Eng., A.M.I.C.E.

A HELICAL flight of stairs (*Fig.* 1) rises 11 ft. between the basement and ground floors of a new office building at Ealing, Middlesex, and has twenty-two 6-in. risers. A landing is provided 3 ft. 6 in. above the basement floor because the fire regulations do not permit a flight of more than fifteen steps. Above the landing the waist of the stairway is 8 in. thick but below the landing the waist is thicker to maintain an unbroken soffit. The flight turns through 360 deg. The imposed load was assumed to be 100 lb. per square foot of area in plan. Two methods were used for analysis. In one method it was assumed that the structure was freely supported, and in the other it was assumed to be fixed at the ends. The methods are described and compared in the following.

Analysis of a Freely-supported Flight.

The notation used and a diagram of the stairs are shown in *Fig.* 2. Assuming that there are no vertical bending moments at the ends of the flight, the basis of the analysis is the formulæ given by Mr. A. H. Mattock[1] which have been adjusted so that the origin is at C midway along the flight. From this point the angle θ measured in a clockwise direction is positive and is negative in an anti-clockwise direction.

Fig. 1.

The angle of rise ϕ is constant and $\tan\phi = \frac{h}{R_2\beta}$, in which h is the effective height of the stair and β is the effective angle through which it turns; γ is 30 deg. In the following formulæ $\alpha = \theta - \gamma$ and $\delta = \theta + \beta/2$. At any point O in the flight,

Vertical moment: $$M_{ro} = wR_1R_2\frac{\beta}{2}\sin\alpha + wR_1^2(1 + \cos\alpha) - \frac{Hh}{\beta}\delta\sin\theta \quad . \quad (1)$$

Lateral moment:

$$M_{no} = \left[wR_1R_2(\cos\alpha - \theta) - \frac{Hh}{\beta}\delta\cos\theta - wR_1^2\sin\alpha\right]\sin\phi - HR_2\sin\theta\cos\phi \quad . \quad (2)$$

Torsion:

$$T_o = \left[wR_1R_2(\cos\alpha - \theta) - \frac{Hh}{\beta}\delta\cos\theta - wR_1^2\sin\alpha\right]\cos\phi + HR_2\sin\theta\sin\phi \quad . \quad (3)$$

Thrust normal to the tangent: $P_{no} = -H\sin\theta\cos\phi - wR_1\theta\sin\phi$. (4)

Shearing force across the waist of the stairs:

$$S_{no} = wR_1\theta\cos\phi - H\sin\theta\sin\phi \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Radial horizontal shearing force: $S_{ho} = H\cos\theta$ . . . . . . (6)

The vertical reactions at the supports are $\frac{w}{2}R_1\beta$. Also $\bar{x} = \frac{2R_1}{\beta}\sin\frac{\beta}{2}$ and $x = R_2\sin\left(\frac{\beta}{2} - 90 \text{ deg.}\right)$. Then $Hh = W(\bar{x} + x)$ and $M_{ho} = HR_2\sin\gamma$.

The values for these stairs are as follows. $\beta = 300$ deg. The internal radius is 3 ft. and the width 4 ft. Therefore $R_2 = 5$ ft. and the centre-line of the loads is $\frac{2}{3}\left(\frac{7^3 - 3^3}{7^2 - 3^2}\right) = 5{\cdot}26$ ft. $= R_1$ from the axis of the helix. The effective height h is the actual height minus one riser and is 10 ft. 6 in. The reactions occurring at the ends of the flight are 13,800 lb., $H = 14{,}040$ lb., and $M_{ho} = 35{,}100$ ft.-lb. Substituting these values in equations (1) to (6) the principal moments, forces, shears, and torque can be calculated and are shown by the full lines in *Fig.* 3.

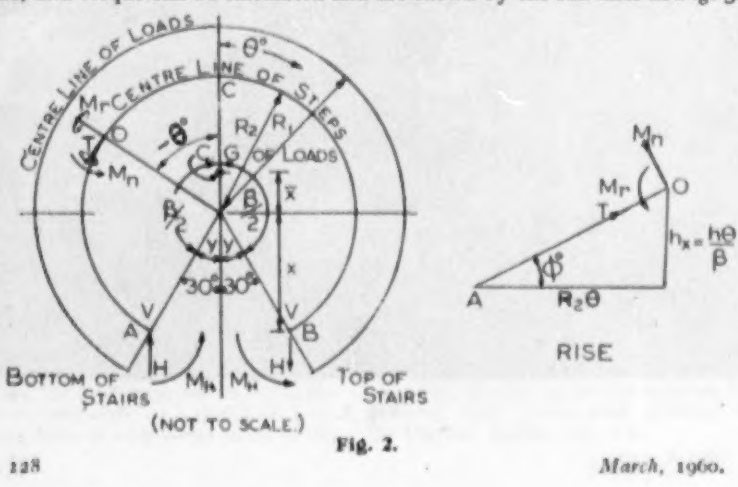


Fig. 2.

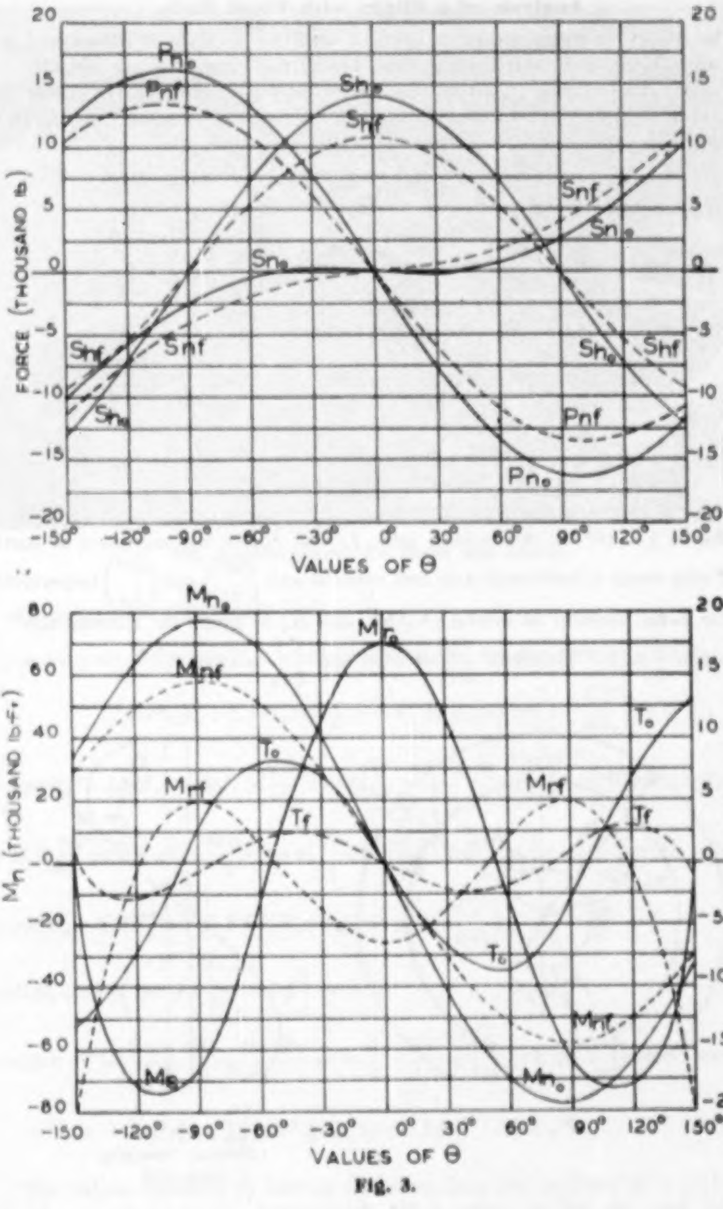

FORCE (THOUSAND lb)
VALUES OF Θ
Pne
Pnf
Sne
Snf
She
Shf
M_n (THOUSAND lb-ft)
M_r & T (THOUSAND lb-ft)
Mne
Mnf
Mre
Mrf
Te
Tf

Fig. 3.

Analysis of a Flight with Fixed Ends.

The theory of strain-energy is applied, and the notation is shown in *Fig.* 4. The origin is point C and bending moments to the right of C are considered to be positive when acting in an anti-clockwise direction when viewed along their axes towards point O. To the left of C all signs are reversed. If M_v is the bending moment acting in a tangential plane at C, then

$$M_{rf} = M_v \cos\theta + HR_2\theta \sin\theta \tan\phi - wR_1^2(1 - \cos\theta) \quad . \quad . \quad (7)$$

$$M_{nf} = (M_v \sin\theta - HR_2\theta \cos\theta \tan\phi + wR_1^2 \sin\theta - wR_1R_2\theta) \sin\phi - HR_2 \sin\theta \cos\phi \quad . \quad (8)$$

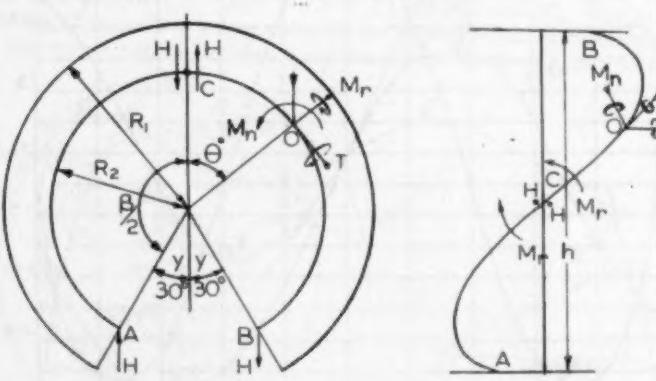
$$T_f = (M_v \sin\theta - HR_2\theta \cos\theta \tan\phi + wR_1^2 \sin\theta - wR_1R_2\theta) \cos\phi + HR_2 \sin\theta \sin\phi \quad . \quad . \quad (9)$$

and the following strain-energy equations may be applied:

$$\frac{\delta U}{\delta m} = 0 = \int_0^{\frac{\beta}{2}} \left(\frac{\delta M_{rf}}{\delta M} \cdot \frac{M_{rf}}{EI_1} + \frac{\delta M_{nf}}{\delta M} \cdot \frac{M_{nf}}{EI_2} + \frac{\delta T_f}{\delta M} \cdot \frac{T_f}{CJ}\right) R_2 \, d\theta \quad . \quad (10)$$

$$\frac{\delta U}{\delta H} = 0 = \int_0^{\frac{\beta}{2}} \left(\frac{\delta M_{rf}}{\delta H} \cdot \frac{M_{rf}}{EI_1} + \frac{\delta M_{nf}}{\delta H} \cdot \frac{M_{nf}}{EI_2} + \frac{\delta T_f}{\delta H} \cdot \frac{T_f}{CJ}\right) R_2 \, d\theta \quad . \quad . \quad (11)$$

in which C is the coefficient of rigidity $= 1{\cdot}2 \times 10^6$ lb. per square inch, E is the elastic modulus $= 3 \times 10^6$ lb. per square inch, I_1 and I_2 are the moments of inertia of one step about a horizontal axis and vertical axis $\left(\frac{bd^3}{12}\right)$ and $\left(\frac{db^3}{12}\right)$ respectively, J is the polar moment of inertia (K_2b^3d), and K_2 as given by Timoshenko[2] is $0{\cdot}31$ when $\frac{d}{b} = 6$. Therefore $\frac{CJ}{EI_1} = 1{\cdot}49$ and $\frac{CJ}{EI_2} = 0{\cdot}0415$.



PLAN (NOT TO SCALE) ELEVATION LOOKING TOWARDS C FROM AB

Fig. 4.

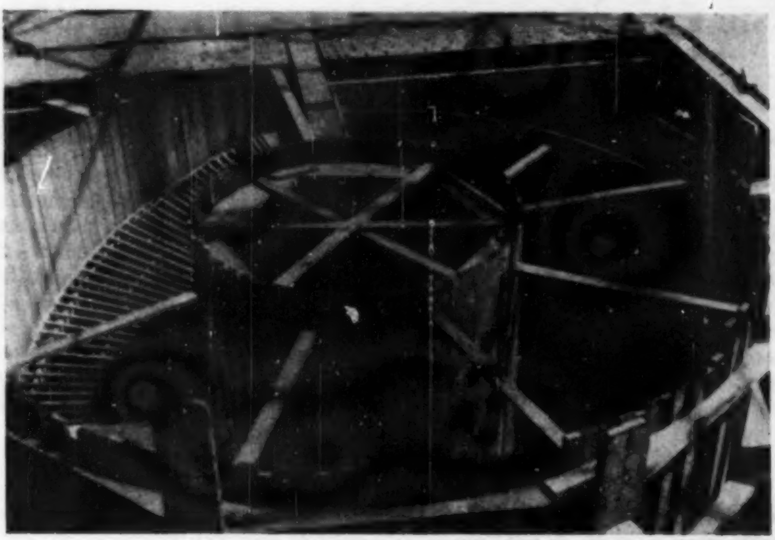


Fig. 5.—Shuttering for Soffit and Edges.

From equations (10) and (11)

$$\frac{CJ}{EI_1}[(m+0{\cdot}5\sin 2\theta)(M_v+wR_1^2)-wR_1^2\sin\theta-KHR_2\tan\phi]+s[m(M_v+wR_1^2)$$
$$+nwR_1R_2+KHR_2\tan\phi]+HR_2\sin\phi\cos\phi\, m\left(1-\frac{CJ}{EI_2}\right)=0 \quad . \quad (12)$$

$$\frac{CJ}{EI_1}\left[nwR_1^2-K(M_v+wR_1^2)+\frac{HR_2}{2}\tan\phi\left(\frac{\theta^3}{3}-\frac{\theta^2\sin 2\theta}{4}-K\right)\right]+s\left[K(M_v+wR_1^2)\right.$$
$$\left.+wR_1R_2(\theta^2\sin\theta+2n)+\frac{HR_2}{2}\tan\phi\left(\frac{\theta^3}{3}+\theta^2\frac{\sin 2\theta}{4}+K\right)\right]+\left(s-\frac{CJ}{EI_2}\right)[m(M_v+wR_1^2)$$
$$+nwR_1R_2+KHR_2\tan\phi]+KHR_2\sin\phi\cos\phi\left(1-\frac{CJ}{EI_2}\right)$$
$$+mHR_2\cos^2\phi\left(\tan\phi+\frac{CJ^2}{EI_2}\cot\phi\right)=0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

in which $K = \frac{\theta\cos 2\theta}{4} - \frac{\sin 2\theta}{8}$; $m = \frac{\theta}{2} - \frac{\sin 2\theta}{4}$; $n = \theta\cos\theta - \sin\theta$; and

$$s = \cos^2\phi + \frac{CJ}{EI_2}\sin^2\phi.$$

The values obtained by solving equations (12) and (13) are $H = 10{,}800$ lb. and $M_v = -6500$ ft.-lb. Substituting these values in (7), (8), and (9) and

Fig. 6.—The Reinforcement and the Shutters for the Risers.

simplifying, the final equations for the bending and twisting moments on the flights when its ends are fixed are

$$M_{rf} = 22{\cdot}56 \cos\theta + 21{\cdot}7\theta \sin\theta - 29{\cdot}07.$$
$$M_{nf} = -41{\cdot}8 \sin\theta - 8{\cdot}1\theta \cos\theta - 10{\cdot}3\theta.$$
$$T_f = 41{\cdot}02 \sin\theta - 20{\cdot}14\theta \cos\theta - 25{\cdot}66\theta.$$

These quantities are shown by the broken lines in *Fig.* 3.

Substitution of the numerical value for H in (4), (5), and (6) gives equations for shearing and other forces when the ends of the flight are fixed. These quantities are shown by the broken line in *Fig.* 3. The reinforcement required to resist bending and shearing forces at various sections are calculated by the common methods.

The resistance to torsion of a rectangular member T_r is given by Saint-Venant as $\dfrac{b^2d^2q}{1{\cdot}8b + 3d}$, and in this case, in which $b = 8$ in., $d = 48$ in., and $q = 125$ lb. per square inch, $T_r = 9700$ ft.-lb. The maximum twisting moment T_f (*Fig.* 3) is 13,000 ft. lb., and extra reinforcement is provided in accordance with the formula given by Prof. W. T. Marshall and Mr. N. R. Tembe. It is clear from the graphs that less reinforcement is used if the stairway is analysed by the second method, the only moments which are greater being the fixing moments at the ends of the flight. Part of the shuttering of the stairway is shown in *Fig.* 5 and the arrangement of the reinforcement in *Fig.* 6.

The architects are Messrs. Lionel H. Fewster & Partners. The stairs were designed by the British Reinforced Concrete Engineering Co., Ltd., who collaborated with the author in preparing this article. The general contractors were Messrs. F. G. Minter, Ltd.

(1).—A. H. Mattock. Design and Construction of a Helical Staircase. "Concrete and Constructional Engineering", March, 1957.

(2).—S. Timoshenko and P. M. Lessells, Applied Elasticity. W.T.N.S. Press, Pittsburgh.

Nomograms for the Design of Beams and Slabs by the Load-factor Method.—II.*

By J. C. STEEDMAN.

Rectangular Beams with Reinforcement in Tension and Compression.

NOMOGRAMS Nos. 4A, 4B and 5 on pages 134 to 136 apply to rectangular beams with reinforcement in tension and compression in accordance with the load-factor method as recommended in British Standard Code of Practice No. 114 (1957). The symbols are as used in the Code with the following additions: The percentages of reinforcements in tension and compression are denoted by r_{st} and r_{sc} respectively. Separate charts are provided for cold-worked bars with $p_{st} = 30{,}000$ lb. per square inch and $p_{sc} = 23{,}000$ lb. per square inch (Nomogram No. 4A); mild-steel bars, $p_{st} = 20{,}000$ lb. per square inch and $p_{sc} = 18{,}000$ lb. per square inch (Nomogram No. 4B); and other combinations of stresses in the reinforcement (Nomogram No. 5). The equations from which the nomograms are derived are

$$\frac{M}{bd_1^2} = \frac{p_{cb}}{4} + \frac{p_{sc}r_{sc}}{100}\left(1 - \frac{d_2}{d_1}\right) \quad \text{and} \quad r_{st} = \frac{r_{sc}p_{sc}}{p_{st}} + \frac{100p_{cb}}{3p_{st}}.$$

The method of using the nomograms is explained in the examples which follow, and the operations required in each example are shown on the nomogram.

EXAMPLE No. 4.—A rectangular beam 9 in. wide and 30 in. deep is subjected to a bending moment of 3,000,000 in.-lb. Determine the required cross-sectional areas of reinforcement in tension and compression if $p_{st} = 30{,}000$ lb. per square inch, $p_{sc} = 23{,}000$ lb. per square inch, and $p_{cb} = 1000$ lb. per square inch. The centre of the reinforcement is 2 in. from each face.

$$\frac{M}{bd_1^2} = \frac{3{,}000{,}000}{9 \times 28^2} = 425; \; \frac{d_2}{d_1} = \frac{2}{28} = 0{\cdot}072.$$

From Nomogram No. 4A, $r_{st} = 1{\cdot}74$ per cent. and $r_{sc} = 0{\cdot}82$ per cent. Therefore

$$A_{st} = \frac{1{\cdot}74}{100} \times 28 \times 9 = 4{\cdot}39 \text{ sq. in. and } A_{sc} = \frac{0{\cdot}82}{100} \times 28 \times 9 = 2{\cdot}07 \text{ sq. in.}$$

EXAMPLE No. 5.—Calculate the cross-sectional area of reinforcement for the beam in example No. 4 if the stresses are $p_{st} = 27{,}000$ lb. per square inch, $P_{sc} = 18{,}000$ lb. per square inch, and $p_{cb} = 1250$ lb. per square inch.

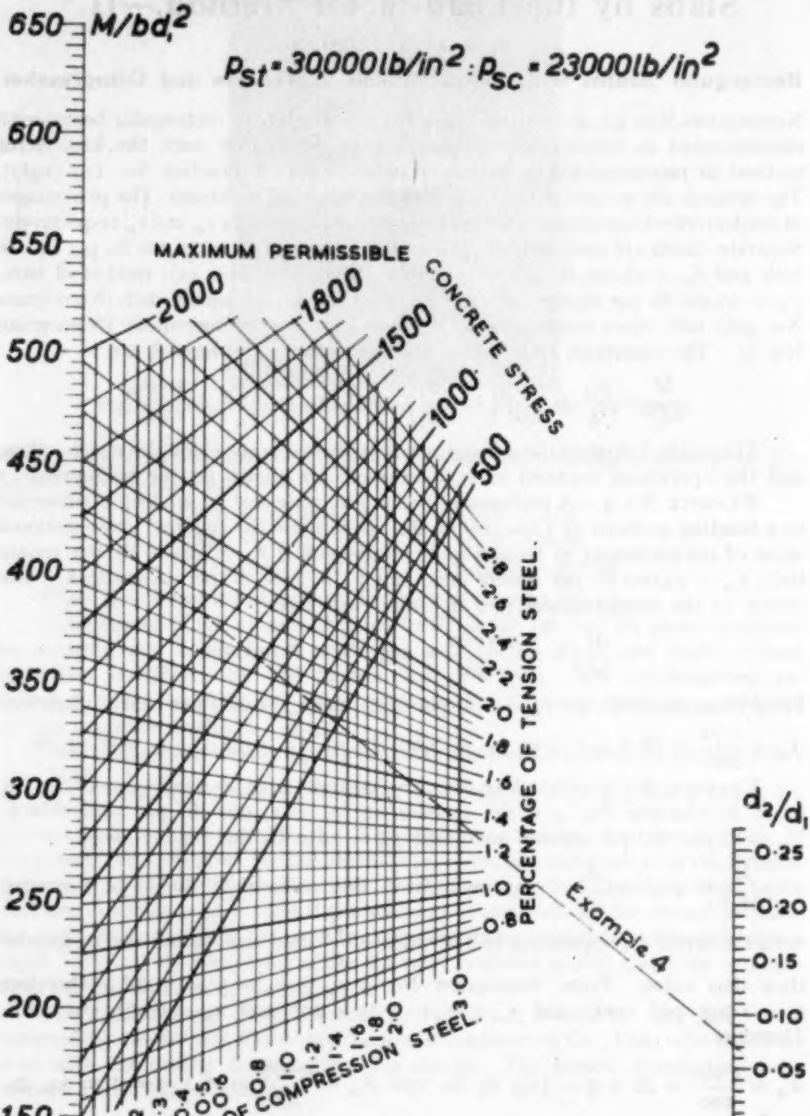
$\frac{M}{bd_1^2} = 425$ and $\frac{d_2}{d_1} = 0{\cdot}072$ as before. The value of p_{sc} should be compared with the stress corresponding to the value of $\frac{d_2}{d_1}$ used and should not be greater than that value. From Nomogram No. 5 $r_{st} \times p_{st} = 5{\cdot}39 \times 10^4$; therefore $r_{st} = 1{\cdot}995$ per cent. and $r_{sc} \times p_{sc} = 1{\cdot}21 \times 10^4$ and $r_{sc} = 0{\cdot}675$ per cent. Therefore

$$A_{st} = \frac{1{\cdot}995}{100} \times 28 \times 9 = 5{\cdot}03 \text{ sq. in. and } A_{sc} = \frac{0{\cdot}675}{100} \times 28 \times 9 = 1{\cdot}70 \text{ sq. in.}$$

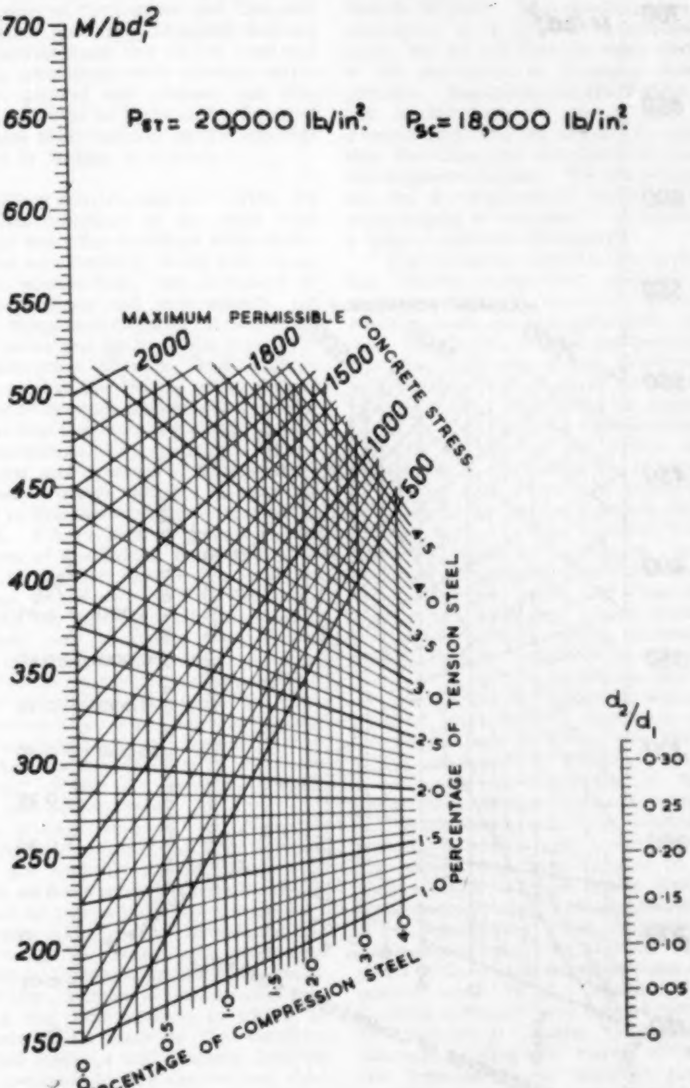
(To be continued.)

* Continued from February number.

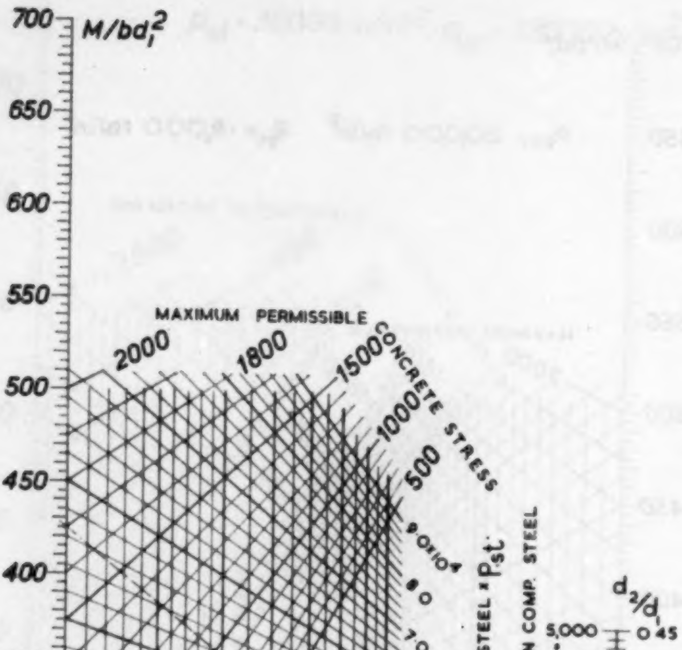
No. 4A.—DESIGN OF RECTANGULAR BEAMS BY THE LOAD-FACTOR METHOD.
Reinforcement in Tension and Compression.



No. 4B.—DESIGN OF RECTANGULAR BEAMS BY THE LOAD-FACTOR METHOD.
Reinforcement in Tension and Compression.



No. 5.—DESIGN OF RECTANGULAR BEAMS BY THE LOAD-FACTOR METHOD.
Reinforcement in Compression and Tension.



Canadian Views on Concrete in Britain.

SEVENTY engineers and architects from Canada visited this country at the end of January and beginning of February under the auspices of the Cement and Concrete Association, and they attended lectures and demonstrations and visited sites and works in connection with concrete structures in general and precast and prestressed concrete in particular. Some of them gave their opinion on the concrete industry in Britain as follows.

"Building construction in London can hardly be described to be other than fabulous; new office buildings forty stories in height and covering acres, new hotels eighteen stories high, the multitude of housing schemes and new schools, the modern hangars at Gatwick, to name only a few, strike one again of the vitality of the construction industry. In Canada we have regarded the British architect and engineer to be very conservative persons, but this trip has certainly dispelled such false impressions. Our special interest was in precast and prestressed concrete construction. British industry is more advanced in this line than its counterpart in Canada. I have been impressed by the eagerness of the British construction industry to develop new design and construction techniques." (A City Engineer.)

"With the high strengths of concrete and steel now available, the plentiful supply of aggregates in Britain, and rigid control of the product, it now appears that precast and prestressed concrete can compete economically with conventional concrete construction for works having long spans and for construction in inclement weather." (Dean of a Faculty of Engineering and Architecture.)

"In precast concrete, we observe a large variety of roof and floor systems which are basically similar, whereas in Canada we have tended to standardise on the best systems and improve them for a wider use. The general acceptance and wider usage of precast and prestressed concrete in Britain is probably the reason for the larger number of systems available. One of the main reasons to which we attribute this trend is the excellent practical research and training facilities made available by the Cement and Concrete Association. We regret that no comparable facilities are available to the concrete industry as a whole, either in Canada or the U.S.A. The development of transit mix (ready-mixed) concrete is more advanced in Canada and the U.S.A. than in Britain. This condition may be attributed to a greater congestion of roads, but we feel that the main obstacle is the resistance to changing existing practice. Regarding concrete finishes and the manufacture of precast and prestressed products, we are of the opinion that the Canadian standards of quality are somewhat higher. We are pleased to see the development of economical art work largely in concrete." (President of a precast-concrete Company.)

"The Canadian construction industry has keener competition between the various structural materials available, namely wood, glu-lam (plywood), rolled steel, cast-in-situ, precast and prestressed concrete. Because of the development simultaneously in Canada of all these materials there has been no concerted effort on the whole of the construction industry to explore the design possibilities and in particular the application of precast and prestressed concrete to buildings as has been the case in Britain. Recent works of the London County Council generally are of a strong, vigorous character, dependent on form and texture rather than on colour and shadow for architectural emphasis. Prestressed precast concrete seems further advanced in Britain than in Canada from the standpoint of using space frames, multiple-story buildings, and the more varied use of small prefabricated structural units. Design methods involving new theories, model studies, and computer analysis seem to be used more and be further advanced in Britain." (An architect.)

"In contrast to some new construction techniques, such as the tower crane for high structures, there appears to be a preponderance of hand labour (two men doing one man's job) and a haphazardness in site organisation. The site planning of Roehampton (flats) is superb. As opposed to the distant impressiveness of this project much of the workmanship in finishing left much to be desired compared to standards in Canada. The design of the main building and 'finger' of Gatwick Air Terminal is an excellent piece of architectural design. Details and finishes are consistently well handled throughout." (An architect.)

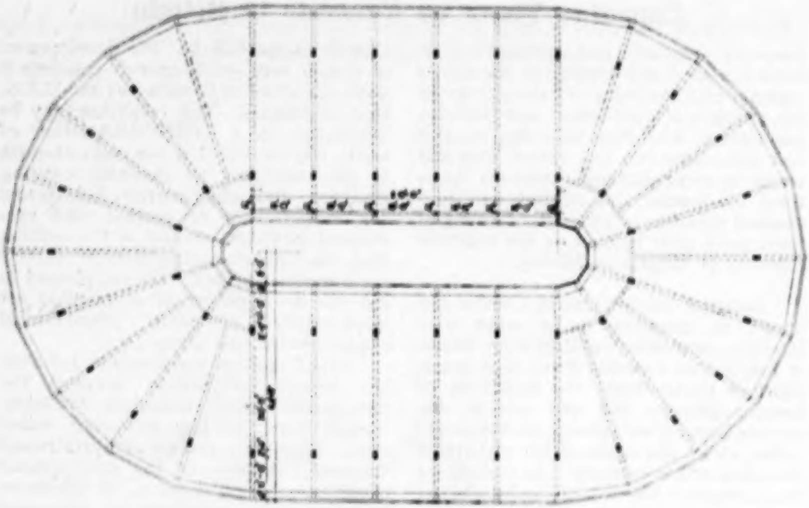


FIG. 2.—PLAN.

FIG. 3.—CROSS SECTION.

Car Park at Bristol. *(See facing page.)*

A Multiple-story Car Park in Bristol.

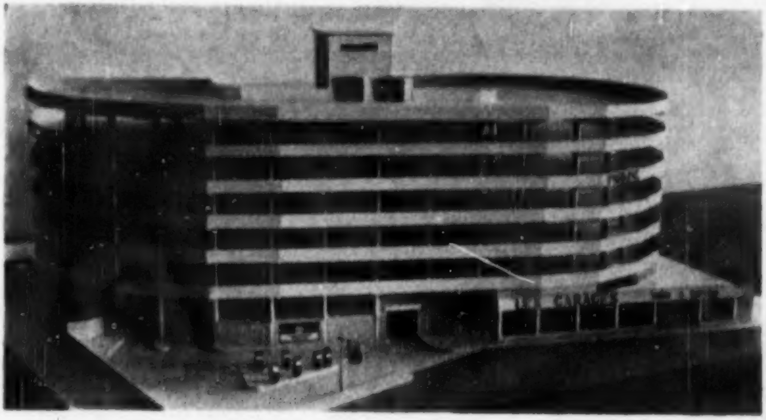


Fig. 1.

A PARKING garage (*Fig.* 1) of six stories is being built at Bristol in the form of a ramp of reinforced concrete supported on reinforced concrete columns and beams and precast piles. The structure is oval on plan, the overall dimensions being about 209 ft. by 129 ft. The ramp is 56 ft. wide. At the centre of the building are two lifts and two staircases by means of which the drivers of the cars will travel to and from the ground floor.

The ramp is supported by columns 1 ft. by 2 ft. in cross section at about 16 ft. centres in the direction of the ramp and in pairs at 32 ft. centres across the ramp. Transverse beams supporting the floor slab span between the columns and cantilever 11 ft. beyond them at both sides of the ramp. Beams in the direction of the ramp are provided only at the inner line of columns at the ends of the building.

The gradient of the ramp at the sides of the building is 1 in 32 and the average gradient at the curved ends is 1 in 46. The ramp has a slope of 1 in 26 towards the inner edge, where rainwater outlets are provided. About 60 per cent. of the area of the ramp will be available for cars and will have a capacity of about 540 cars. A typical plan is shown in *Fig.* 2, and a typical cross section in *Fig.* 3.

The ground floor will have two petrol stations and several shops and showrooms, one of which will contain a turntable for cars. The joint consulting engineers are Mr. E. N. Underwood and Messrs. G. C. Mander & Partners, the consultant architect is Mr. R. Jelinek-Karl, and the main contractors are Messrs. William Cowlin & Son, Ltd. The structure is being built for a company controlled by Mr. M. Weston, of London.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Settlement of Buildings, by S. R. Rosenak. South Dorset Technical College, Newstead Road, Weymouth. March 22; 7.30 p.m.

Work Study in the Building Industry, by K. C. Symons. College of Art, Green Lane, Derby. March 23; 7.15 p.m.

Practical Formwork Design and Construction of Concrete, by J. G. Richardson. College of Technology, Howard Street, Rotherham. March 24; 7.15 p.m.

Prevention of Accidents in the Building Industry, by J. A. Hayward. Cleveland Technical College, Corporation Road, Redcar. March 29; 7.15 p.m.

Prestressed Concrete, by H. Kaylor, Municipal Technical College, Blackburn. March 31; 7.15 p.m.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", March, 1910.

A Retaining Wall 50 ft. High.

A VERY high retaining wall in Los Angeles is shown in the illustration. "The wall has a length of 600 ft. and is 52 ft. 8 in. high at maximum height, with a minimum of 13 ft. The wall is surmounted by a 5-ft. parapet. The foundation at maximum height of wall is 14 ft. wide; the wall ranges in width from 8 ft. 1 in. to 3 ft. wide at the base of the parapet. The specifications called for construction in 40-ft. sections, no two adjoining sections to be constructed simultaneously. Alternating expansion and contraction joints are provided for. The section method of construction required frequent removal of the contractor's plant and two sets of workmen were employed, one for mixing and spreading concrete, the other on the forms and back-filling the wall with a 6-in. layer of coarse gravel. The concrete of one section was allowed to set before another section was begun. The expansion joints are reinforced with pilasters (at 80 ft. centres). The contraction joints were formed midway between the pilasters."

Courses for Concrete Gangers.

THREE full-time courses for concrete gangers have been prepared by a joint committee of the London Master Builders Association and the Federation of Civil Engineering Contractors in collaboration with the Cement and Concrete Association. The object of the courses is to provide simple and practical instruction for the concrete ganger or potential ganger. The courses will be conducted by the Cement and Concrete Association at Wexham Springs, Bucks., and each course will occupy two weeks with a break of a fortnight between each week. The first course will be from April 25 to April 29 and May 16 to May 20; the second course is from May 2 to May 6 and May 23 to May 27; and the third course is from May 9 to May 13 and May 30 to June 3.

Details of the courses can be obtained from the Education Department of the London Master Builders Association at 47 Bedford Square, London, W.C.1.

Design of a Helical Stair Built in 1908.

THE contractors for the helical stair built in 1908 at the Franco-British Exhibition, and described in our number for January, 1960, were the Yorkshire Hennebique Contracting Co., Ltd.

Precast Ribbed Floor Slabs.

A NEW type of ribbed floor slab has been used in the construction of a three-story office building 228 ft. long by 30 ft. wide for the Atomic Energy Authority at Winfrith Heath, Dorset. The external frame (*Fig.* 1) includes eighty-eight reinforced concrete columns which were precast in lengths of 33 ft., which is the height of the building.

The floor panels are each 15 ft. by 12 ft., and have ribs in two directions forming 18-in. squares. The total thickness of each panel is 7½ in., which includes the runners for fixing the ceiling and a slab 1½ in. thick. The slab is reinforced with lightweight steel mesh having apertures 3 in. square. Therefore holes for services can be made at any part of the slab between the ribs. Each panel weighs 3½ tons and was lifted by a crane with tackle attached at four points (*Fig.* 4).

The junction of the panels with the columns is a combination of a mechanical and cast-in-place concrete joint (*Fig.* 2).

At each floor a strip of steel plate ⅜ in. thick and 4½ in. deep, and with a 1-in. twisted deformed bar welded to the top and bottom edges, projects 10½ in. from opposite faces of the columns. In the underside of the ribs of the precast panels there are slots 2 in. to 3 in. wide into which the plates fit when a panel is lowered into position. The weight of the panel is transferred to the plate by direct bearing on suspension bars embedded in the

Fig. 2.—Connection Between Column and Floor Panel.

Fig. 1.—The Frame During Construction.

Fig. 3.—Erecting a Column.

concrete above and at the sides of the slot. The slots are filled with 1 : 1 : 2 concrete which is injected through openings in the $1\frac{1}{2}$-in. slab above the slots. When the filling has hardened the shearing forces are transmitted partly by bearing and partly by adhesion of the filling. Connections of this type have been tested by applying the equivalent of loads exceeding the total dead and imposed load on one side and the dead load only on the other side.

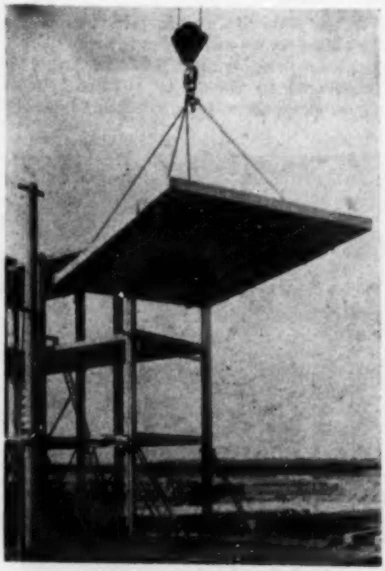


Fig. 4.—Erecting a Floor Slab.

The columns are at intervals of 12 ft. in four rows along the length of the building. Each outer row is 12 ft. from the inner rows, which are 6 ft. apart. Each floor panel is supported by four columns (*Figs.* 1 and 3) and cantilevers 3 ft. beyond the columns in the inner rows, thereby forming half the floor of the central corridor.

The stairs comprise separate precast members of inverted L-shape which were placed by one man on to a pair of beams, which form the up-stand carrying the balustrade.

The time taken for the erection of the precast frame was twenty-five days. It is claimed that a column can be erected in twenty minutes, and a floor slab in ten minutes, by lifting them directly from a lorry and placing them in their final position. The designer of the structure was Mr. Stephen Revesz, and the main contractors were the Turriff Construction Corporation, Ltd. The Modular Concrete Co., Ltd., were the contractors for the reinforced concrete work and also made the precast members.

Large Bored Foundation Piers.

A U.S.A. machine for drilling foundation piers was used for the first time in this country on the site of a 16-story office building now being erected in Euston Road and Gower Street, London. The machine (*Fig.* 1) comprises a chassis, with hydraulic stabilising jacks, surmounted by a retractable jib 78 ft. long which is in a horizontal position during transit.

At the position of each pier the machine is levelled by the jacks and the jib is raised to a vertical position. Within the jib is a square steel kelly-bar which is rotated by means of a rotatable table operated by a diesel engine. A hard-steel auger of the diameter required is attached to the bottom of the kelly-bar by a quick-release pin, and when it is rotated it bores into the ground at a speed depending on the hardness of the material. The auger is raised to the surface at short intervals, and by a rapid rotational movement the excavated material is flung off the blades and falls around the perimeter of the hole (*Fig.* 2). When the hole has been drilled to the depth required the auger is replaced by an under-cutting tool, which flares out the bottom of the hole to a greater diameter, thus providing an increased bearing area for the pier. Upon completion of the boring operation reinforcement, if required, is lowered into the hole and the entire excavation is filled with concrete up to the level of the underside of the pile-cap or other foundation structure. The machine can drill holes up to 8 ft. diameter to great depths and the rate of boring, which depends on the size of the auger, may be several feet per minute.

At the Euston Road and Gower Street site there are 145 piers of diameters from 21 in. to 60 in., and penetrating to 60 ft. into firm London clay. Reinforcement is provided in the top of the larger piers to act as splice-bars. In the smaller piers reinforcement is provided from the top to within 20 ft. of the bottom. A steel lining-tube was used temporarily at the top of each hole to support the gravel which overlies the clay. Ready-mixed concrete was used as the main contractors' mixing plant had not been erected.

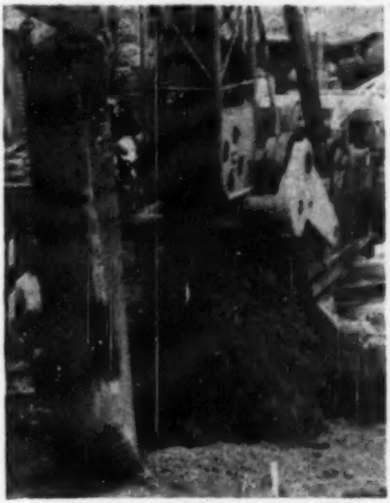


Fig. 2.

The machine was operated by McKinney Foundations, Ltd., who are associated with Messrs. John Laing & Son, Ltd., the main contractors for the building.

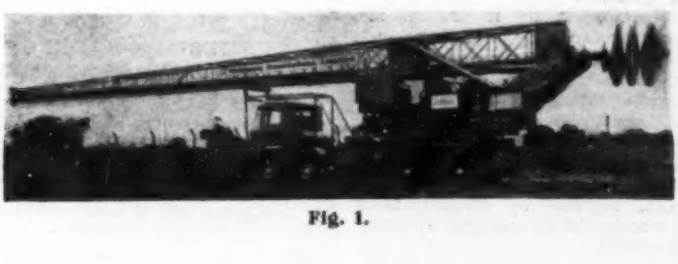


Fig. 1.

"Towards the Robot."

MR. R. M. AMODIA comments as follows on the "Editorial Notes" in our number for January, 1960.

Very rightly, you have condemned the trend towards over-specialisation at the cost of broader outlook and liberal education. In my view, the outlook of industry is much more responsible for this trend. It is apparent, for example, that a man working on, say, steel structures finds it difficult to get employment on concrete design, and a man working on highways finds it difficult to get employment on water-supply and sewerage schemes. What else could be the outcome if not over-specialisation ?

This attitude does create robots, and, in the long run, robots without real efficiency.

[A Robot is a machine in the form of a man in which are installed mechanisms that enable it to perform certain functions, but it cannot think for itself. Our article was a plea that men should be educated so that they can think logically as well as apply automatically the principles and formulæ that have been instilled into them at a technical school. At present this is seldom the case, as this and other articles published in this journal have clearly shown. The evil is not confined to civil and structural engineering. We have frequently quoted the misgivings of leaders of other professions on the lack of education and general knowledge of technologists. "The Builder" for January 22, 1960, page 164, quotes a report emanating from the Welsh School of Architecture in which it is stated that the students are in general of a lower standard of education than is required in any comparable profession and often behave like children, and that the staffs often perform functions for which they are not qualified.

Specialisation to-day is unavoidable, but this is no reason why specialists should not be educated men and women capable of logical thought and of an appreciation of the arts. We suggest that our correspondent, and others in a like position, make a serious effort to realise that their work is part only of their lives, and endeavour to widen their interests in intellectual matters by making their friends among people engaged in other pursuits.—ED.]

Plastic Lining for Column Shutters.

THE illustration shows the column shutters used in the construction of a bridge on the St. Albans by-pass road. The columns are 14 ft. high and 4 ft. 6 in. diameter, and were concreted to the full height in four hours with the aid of a skip

on a mobile crane. The timber is lined with polystyrene plastic sheet made by Monsanto Chemicals, Ltd., in order to produce a smooth surface. The shutters are removed twenty-four hours after the concrete is cast, and concrete does not adhere to the lining. The contractors are Messrs. Holland & Hannen and Cubitt, Ltd.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 5d. a word: minimum, 12s. Situations Vacant, 6d. a word: minimum, 15s. Other miscellaneous advertisements, 6d. a word: minimum, 15s. Displayed advertisements, 40s. per column inch. Box number 1s. extra.

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SITUATIONS VACANT. South London consulting engineers require experienced reinforced concrete engineers, designer-draughtsmen, and detailers. Applicants should have a minimum experience of three years in position applied for. Salary commensurate with experience and ability. Luncheon vouchers, and five-days' week. Please write, giving details of age, qualifications, and experience, to LEONARD & GRANT, 344 South Lambeth Road, London, S.W.8.

SITUATIONS VACANT. Consulting engineers have vacancies for reinforced concrete detailers, and designer-detailers. Salaries in accordance with experience. Luncheon vouchers. Apply in writing to BYLANDER, WADDELL & PARTNERS, 169 Wembley Park Drive, Wembley, Middlesex.

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SITUATIONS VACANT. Small structural engineering office, London, W.C.1 area, with extensive practice, requires additional assistants. Capable men from graduateship upwards required for high-class miscellaneous work generally, foundations, dangerous structures, steel, concrete and timber, and able to work with minimum supervision. Reply in confidence, with full particulars, and salary required, to Box 4634, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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SITUATIONS VACANT. Reinforced concrete engineers require reinforced concrete designer-detailer and detailing draughtsmen for reinforced concrete work. Write, giving age, experience, and salary required, to CONSIDERE CONSTRUCTIONS LTD., 72 Victoria Street, London, S.W.1.

SITUATION VACANT. Prestressed concrete supervisor wanted by well-established company for works on western outskirts of London. Applicant must have wide experience of various systems, and be able to deal with all aspects of labour utilisation, production and quality control. The post is pensionable, and offers excellent prospects to the right man. Existing staff have been notified of the vacancy. Please write, giving in confidence, full details of experience, age, and salary required, to Box 4649, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Senior reinforced concrete designer/detailers and experienced detailers required in Hammersmith office of consulting engineers. High salaries and good prospects, with interesting work. Five-days' week, pension scheme. Apply, in confidence, with full details of experience, and salary required, to ALAN MARSHALL & PARTNERS, Federal House, 2 Down Place, London, W.6. Telephone Riverside 8771.

SITUATIONS VACANT. Consulting engineer, Westminster, invites applications from detailer/draughtsmen with a minimum of four years' experience in reinforced concrete. Salary scales appropriate to experience and ability. Non-contributory pension and bonus schemes in operation. Holiday arrangements will be honoured. Apply in writing, giving full particulars, to Box 4650, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Designer/detailers for reinforced concrete. Experience in civil engineering an advantage. Salary according to qualifications and experience. Luncheon vouchers are provided. Apply, in writing, to HUSBAND & Co., Consulting Engineers, 70 Victoria Street, London, S.W.1.

SITUATIONS VACANT. Civil engineering department of THE COFFEE CO. (G.B.) LTD., 140 Piccadilly, London, W.1, require reinforced concrete designers and detailers for wide variety of industrial structures. Steelwork experience additional advantage in one of the positions to be filled. Five-days' week. Pension scheme in operation. Write, stating age, experience, and salary required.

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Published by CONCRETE PUBLICATIONS LTD., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
Printed by BUTLER & TANNER LTD., The Selwood Printing Works, Frome and London